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Final Report

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16. Abstract

A three and one-half day workshop was used as a forum to examine the broad field of pavement rehabilitation and related strategies applicable to both highways and airfields. The primary purpose of the workshop was to formulate a proposed research framework aimed at producing optimal solutions to problems of structural rehabilitation. Prepared state of the art papers were presented to document knowledge impinging on the pavement maintenance processes. These papers were used as background material for study by small working groups. A proposed research framework was formulated from an appraisal and evaluation of the workshop deliberations. This report contains the proceedings of the workshop.

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Memorandum

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

TO : Individual Researchers

DATE: November 11, 1974

In reply refer to: HRS-14

FROM : Project Manager, FCP Project 5D, "Structural Rehabilitation of Pavement Systems"

SUBJECT: Transmittal of Research Report No. FHWA-RD-74-60, "Pavement Rehabilitation"

Distributed with this memorandum is the subject report intended primarily for research audiences. This report will be of interest to highway engineers concerned with the many aspects of pavement rehabilitation.

This report contains the proceedings of a three and one-half day workshop. The workshop was used as a forum to examine the broad field of pavement rehabilitation and related strategies applicable to both highways and air fields. The primary purpose of the workshop was to formulate a proposed research framework aimed at producing optimal solutions to problems of structural rehabilitation.

Sufficient copies are being sent to division offices to provide one for each State highway department.

Additional copies are available from the National Technical Information Service (NTIS), Department of Commerce, 5285 Port Royal Road, Springfield, Virginia 22151. A small charge is imposed for copies provided by NTIS.

Thomas Krylowski

Thomas Krylowski

Attachment

PREFACE

In the spring of 1973 the Office of Research and Development, Federal Highway Administration, requested the Highway Research Board to convene a conference to examine the broad field of pavement rehabilitation and related strategies applicable to both highways and airfields and to formulate a proposed research framework aimed at producing optimal solutions to problems of structural rehabilitation.

The Highway Research Board formed an advisory committee to consider how the Board might proceed. The committee recommended a workshop with a four-step progression of activities as the most fruitful method of accomplishing the objectives. First, a number of authors would be invited to prepare and present a comprehensive set of state-of-the-art papers on pavement rehabilitation and related problems and strategies. The rehabilitation problem would then be structured into two categories of subtopics permitting small working groups of individuals to discuss each subtopic and prepare a summary report. Concurrently, a group of consultants to the committee would observe the workshop deliberations and prepare a summary appraisal and evaluation of the proceedings including currently implementable information. Finally, another group of consultants to the committee would evaluate recommendations emanating from the proceedings and formulate a proposed research framework.

A 3-1/2 day workshop was held at the Hotel San Franciscan, San Francisco, California, on September 19-22, 1973. Approximately 75 of the most qualified engineers and scientists available including individuals from highway and transportation departments, federal agencies, industry, trade associations, universities, and research institutions participated. International representation included individuals from Canada, The Netherlands, Great Britain, Sweden, and South Africa.

Prior to the meeting, state-of-the-art papers were distributed to the participants along with information outlining the aims and objectives of the workshop, program, format, method of accomplishment, and instructions to participants.

The workshop proceedings, including the summary evaluation and proposed research framework, were reviewed and approved by the advisory committee. The names of the workshop participants and the advisory committee are included.

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INTRODUCTION

Most of the federal, state, and local agencies that are responsible for highway maintenance face many serious problems. These include aging highways nearing the end of their design life and now beginning to require major rehabilitation to provide for heavier loadings, public demands for higher levels of service, stricter safety requirements, environmental protection requirements, shortages of qualified personnel and rapidly escalating rates for labor, equipment, and materials. State disbursements for maintenance of highways in 1971 alone amounted to $2.1 billion. Similarly the development of larger airports as well as increased aircraft loadings and frequency of operations reflect a parallel pattern to highway experience.

Executive branch recommendations to the U. S. Congress that load restrictions on Interstate highways be raised to permit more economical truck operations, if adopted, may be expected to increase pavement and bridge maintenance costs by 20 percent. It has been estimated that 5 million tons of asphalt, an energy resource, and large quantities of other critical materials are used annually for maintenance purposes. And the policy committee of the American Association of State Highway and Transportation Officials has recommended that the definition of construction be expanded to include pavement overlays. Because maintenance has always been the responsibility of state governments following construction of the facility, this recommendation, if adopted, will involve the federal government in an active role for which it previously had no responsibility.

The implications of these changes and the trend toward use of funds by maintenance that might otherwise be expended on developing and improving transportation facilities make their wise use essential. In particular, maintenance of these systems has taken on great significance to ensure continued structural adequacy together with satisfactory riding quality and adequate skid resistance.

As with other aspects of transport, only limited funds are available for the maintenance functions. Accordingly, it is most important to carefully evaluate existing maintenance methodology, and from this evaluation devise needed research to insure optimum use of available resources in pavement maintenance.

While there are a number of reasons for determining that a pavement requires maintenance, e.g.,

1. structural deterioration leading to a reduction in load carrying capability,
2. inadequate riding quality (excessive roughness),
3. inadequate skid resistance, and
4. surface deterioration (raveling),

only the area concerned with structural rehabilitation will be examined in detail in this report in conformance with the request from the Federal Highway Administration.

To develop the material included herein, a workshop of knowledgeable individuals from the transportation community was convened in San Francisco September 19-22, 1973. The purpose of the workshop was to review the state of the art of pavement rehabilitation and related strategies applicable to both highways and airfields and to formulate a proposed research framework that will produce optimal solutions to the problem of structural rehabilitation of pavement systems. Specific objectives were to

1. Assemble and present pertinent information concerning the rehabilitation of highway and airfield pavements,
2. Appraise and evaluate pertinent information,
3. Identify implementable information that can readily be incorporated into practice, and
4. Identify areas needing further research and formulate a research framework to guide the Federal Highway Administration in the development of their research program in structural rehabilitation of pavement systems.

This report details the results of this workshop and includes the following information:

1. Appraisal and evaluation of current state of the art detailing implementable material that can be put into practice immediately, and

2. A summary of needed research on structural rehabilitation.

The basis for these recommendations is contained in a series of state-of-the-art papers covering various facets of the maintenance process together with reports prepared by the working groups who participated in the workshop. Included is an outline of the workshop format to provide an indication of the methodology used to arrive at the material presented.

PROPOSED RESEARCH FRAMEWORK

L. G. Byrd and F. N. Finn

The Workshop on Pavement Rehabilitation was organized to produce a series of papers designed to (a) establish a framework for pavement rehabilitation, (b) summarize the state of the knowledge of subjects pertinent to pavement rehabilitation, (c) summarize and evaluate the state of the art of pavement rehabilitation as developed by the Workshop, and (d) identify research needs. These latter two reports were designed as companion efforts and were prepared after completion of the Workshop and after reports were received from the 11 working groups composing the Workshop.

The paper by McCullough and Epps contains a comprehensive overview of the available technology related to pavement rehabilitation for both highway and airfield pavements. The report contains some suggestions for needed research regarding specific areas of technology.

The purpose of this report for a research framework is to identify research needs as obtained from the discussions and reports of the working groups and develop a framework for organizing research into a structured research program. Except when noted otherwise, the research needs presented are equally applicable to both highway and airfield pavement rehabilitation.

The Workshop provided for discussions on improvements to the conventional approach to rehabilitation, usually associated with overlays, plus recommendations for needed developments in innovative new design procedures, materials, and construction techniques.

FEDERALLY COORDINATED PROGRAM OF RESEARCH AND DEVELOPMENT IN HIGHWAY TRANSPORTATION

The need for research in the field of pavement rehabilitation has been recognized by the Federal Highway Administration in its federally coordinated program of research and development in highway transportation. While the entire program affects pavement rehabilitation directly or indirectly, that program category designated as category V -- Improved Design to Reduce Costs, Extend Life Expectancy and Ensure Structural Safety, and specifically Project D, Structural Rehabilitation of Pavement Systems, focuses on this problem area.

Project V-D is concerned with the development of improved methods for the structural rehabilitation of damaged pavement systems. The research is concerned with the treatment of existing layered systems with overlay design, with measurement of pavement serviceability, and with use of new materials and design methods.

Other parts of the federally coordinated program that have direct input to pavement rehabilitation research include

I-H. Skid Accident Reduction, where research effort will be directed toward improvements in maintenance of skid-resistant pavement surfaces;

IV-A. Minimize Early Deterioration of Bituminous Concrete, where attention will be given to characteristics and behavior of bituminous materials and mixes related to early deterioration of pavements in service;

IV-B. Eliminate Premature Deterioration of Portland Cement Concrete, where attention will be given to frost damage, uncontrolled longitudinal cracking, and excessive abrasion of the pavement surface;

V-E. Premium Pavements for Zero Maintenance, which will exploit modern materials and design technology to develop premium pavements for warranted uses;

VI-A. Construction and Maintenance Materials, which will identify, evaluate, and promote use of new materials and products for maintenance including patching pavements; and

VI-B. Construction and Maintenance Methods and Equipment, which will identify, evaluate, and promote use of new methods, processes, and equipment for maintenance, including patching and sealing pavements.

GROUP DISCUSSIONS ON RESEARCH NEEDS

As a part of the deliberations of each group at the Workshop, research needs relevant to that group's area were identified and, in some instances, rated in importance. A brief summary of the group discussions follows.

Group I. Condition Surveys

This group identified three areas of needed research:

1. Uniformity in condition survey techniques and procedures. Standardization on a regional or national basis was advocated, with procedural manuals and training centers (such as FHWA regional skid calibration centers) as a part of an implementation program.
2. Objective, mechanistic techniques and equipment. A universal testing vehicle is needed, capable of conducting skid, roughness, and condition surveys simultaneously at highway speeds.
3. Definitive relationship between distress and performance. A method is needed to weigh and combine various pavement conditions into a single distress index with a known relationship to pavement performance.

Group II. Measurement Systems

In its discussion, an inventory and evaluation of known measuring systems were developed. Research needs relative to measuring systems were suggested to include a single instrument capable of measuring all the properties needed for diagnostic studies of pavements. The instrument must be simple to operate and functional at relatively high speeds. Specifically, the group recommended development of instruments based on wave propagation and wave velocity measurements, including laser and photographic measurement of deflection.

Group III. Materials and Techniques

An extensive list of specific research problems was identified by this group. The needs, in order of importance were classified as follows:

1. Rapid repair and construction. Techniques and materials are needed to provide for low-effort, rapid setting repairs that permit quick restoration of traffic to the pavement.
2. Preparation and strengthening of existing structures. Strengthening of subsurface layers and provision of uniform strength of existing structure preparatory to overlaying were stressed.
3. Thinner overlays. Because thickness of overlays is an important consideration in overpass clearances, curbs and gutters, and bridge approaches, as well as in cost and use of materials, development of methods and materials for successful use of thinner overlays is needed.
4. Environmental and energy constraints. Materials and techniques with low energy requirements and with minimal adverse effects on the environment are needed and may be mandated by increasing constraints in these areas.
5. Other. Skid resistance and control of reflection cracking research needs were identified along with development of new or improved cements, concrete mixes, mastics, plastics, precast units, insulating materials, and recycled materials.

Group IV. Strategies

Consideration was given by this group to strategies applicable to network pavement systems as well as to specific pavement sections. In evaluating research needs relative to strategies, the group focused on the need for optimization models and criteria for optimization. Specific criteria given a high rating for research needs by the group were highway user costs, highway economic criteria, and airfield pavement comfort.

Group V. Reflection cracking

Cracks in existing pavements reflecting through new surfacings were considered by this group. Four research needs related to this problem were identified:

1. Mechanistic study. The need exists for a thorough understanding of the mechanisms involved in reflection cracking and its control or prevention. Mechanistic models need to be developed.

2. Criteria and guidelines for field data collection. Ongoing field tests of proposed new remedies require specific direction and standardization of data collection to permit field verification of mechanistic models.
3. Improved strength characteristics of bituminous concrete mixtures. Improved tensile, shear, and elastic properties could reduce or eliminate the reflective cracking problem.
4. Deterioration in overlay at reflection crack. Study is needed to define the mechanisms in the development and progression of deterioration at reflection cracks and to develop preventive or corrective measures.

Group VI. Other Special Problems

Three areas of concern suggested research needs to this group.

1. Rehabilitation of high-volume, high-speed pavements. Specific research needs under these conditions include handling traffic, rapid repair methods, and an overall economic strategy.
2. Thermally cracked pavement. Research is needed to determine the severity of this problem in the United States and to develop rehabilitation strategies, if warranted.
3. Environmental and ecological constraints. Research is needed to develop alternative uses of material resources in view of diminishing availability and environmental restrictions.

Group A. Modification of Structural Design Procedures

Research needs identified by this group are as follows:

1. Performance data on overlaid pavements, to evaluate and adjust proposed models for overlay design; and
2. Methods for expressing existing pavement structural conditions (as determined by rapid, objective measurements on the in situ pavement) in terms acceptable to the original pavement design methodology.

Group B. Layered Elastic Systems

Consideration was given by this group to the use of elastic layered theory in pavement rehabilitation. Research needs of significant priority identified by the group are given below:

1. Failure criteria. Research is needed to delineate what failure parameters (types of distress measured and method of determining magnitude) should be included in a rehabilitation scheme based on elastic layered theory.
2. Applicability to performance model. Research is needed to establish this relationship for elastic layered theory in rehabilitation design.
3. Applicability to cracked pavements. There is a need for technical information on whether the use of a reduced modulus to account for crack discontinuities will yield adequate correlation to measured stresses, strains, or deflections.
4. Material characterizations. Effort is needed to verify lab characterizations with actual field values.
5. Remaining life concept. Much work is needed to better define the methods of determining the remaining life of a pavement system.

Group C. Deflection, Curvature, and Stiffness Based Procedures

Ten overlay design procedures currently in use and based on deflection, curvature, or stiffness were identified and evaluated by the group. Current research is under way, and continuing research is needed to obtain feedback data to validate or improve the procedures and to include such factors as materials differences and environment. Specific areas of research needs identified by this group were

1. Fundamental deflection basin relationships to performance and material properties,

2. Seasonal variations in deflections for adjustment to critical periods (such as spring conditions),
3. Material characterization from laboratory data to field conditions,
4. Use of deflections for various material types such as flexible, rigid, and composite, and
5. Relationship of deflection to vehicle speed.

Group D. Other Procedures

This group dealt with the widely used overlay design practice that depends on the experience and judgment of the designer rather than rational analyses.

1. The definition and design of a data bank to incorporate necessary elements of overlay design, evaluation, and control plus compatibility with total needs of a pavement management system, including ready retrieval capabilities.
2. An economical means of maintaining skid resistance throughout the structural life of a portland cement concrete pavement.
3. An economical means of rehabilitating structurally distressed travel lanes of multilane roadways where passing lanes show no signs of distress.

Group E. Ideal Procedures

Criteria for an ideal model to predict rehabilitation action were discussed by this group. Research identified as necessary to develop the ideal system is needed to provide

1. Capability to predict rutting, cracking, and disintegration from both loading and environmental factors.
2. Equipment and data collection systems, developed to monitor skid resistance, roughness, surface condition, and structural adequacy in a concurrent, rapid, non-destructive manner.
3. Capability to predict changes in the fundamental characteristics (strength, thickness, durability, moisture,) of component materials in the pavement structure over time.
4. Capability to predict variability of performance based on input data.

Other research needs suggested by this group were divided into two categories.

1. Monitoring system needs: (a) continuous recording equipment for traffic types and loads; (b) data storage and retrieval system for pavement history: construction quality control, maintenance, and repairs and costs; (c) incorporation of environmental factors and ecological characteristics; and (d) statistical sampling procedures for input data for model.
2. Predictive model research needs to yield information on (a) changes in roughness, (b) loss of skid resistance, (c) frost problems and severe cold climates, and (d) user costs.

Correlation between distress mechanisms and performance, also requires research.

Summary of Research Needs Identified by Groups

Throughout the foregoing discussion of group reports, there is evidence of a concensus on many fundamental research needs that must be met to deal with the problems of pavement rehabilitation. The research needs can be classified into several basic groups.

Pavement Evaluation Equipment -- Several groups focused on the need for the development of a single standard high-speed testing machine that could operate at highway speeds while measuring and recording all of the properties needed for diagnostic evaluation of pavements.

Pavement Management -- There was consistent concern among the groups over the need for a definitive pavement management program. Such a program would include

1. Standards, criteria, and guidelines for field data collection,

2. Established procedures for relating distress as defined by engineers to performance as expressed by the user, and
3. A data bank model to house and serve in evaluation, design, control, and other elements of pavement management, including strategy for pavement management program planning.

Pavement Overlay Design -- Most groups felt that improved design was dependent on two areas of research:

1. Mechanistic study to better define and understand pavement distress and failure phenomena and to establish distress-performance relationships, and
2. Performance data from field observations of overlaid pavements to support or refute design concepts.

Pavement Repair/Rehabilitation Procedures -- Discussions in this area suggested a concensus on several research needs.

1. Rapid repair capability to reduce roadway occupancy time,
2. Restoration capability to produce a degree of uniformity in existing pavement structure before overlaying, and
3. Thin overlay capability to reduce clearance, drainage and other problems of thick overlays.

Specific Research Projects by Priority

From the general categories listed in the foregoing section, a more definitive list of specific research projects can be developed to reflect the thinking and recommendations of the Workshop participants. This list would be difficult to assemble in order by priority, but does readily separate into at least two groups.

High-Priority Research Projects -- These projects include use of fast-setting, precast, replaceable, renewable elements in the wearing surface of pavements for repair and rehabilitation of localized failures; development of injection, subsealing, bonding, or other procedures and materials to permit restoration of structural capability to distressed pavement sections before overlaying; development of criteria and guidelines for standardization of pavement evaluation field data collection; development of a pavement management program, including a data bank model; and development of a testing machine capable of operating at highway speeds while measuring and recording needed data for pavement evaluation.

Lower Priority Research Projects -- Projects of lower priority include reflection crack control or prevention and study of failure mechanisms and methods of eliminating reflection cracks or controlling and negating the problems they cause; study of materials, strength characteristics of mixes, and design concepts leading to thinner overlays; environmental and energy constraint accommodation with study of materials and techniques leading toward selections having minimal effect on the environment and low energy requirements; and design procedures with study to continue evaluation and refinements of existing design concepts and development of new design systems. Study should include layered elastic theory, deflection, curvature, stiffness and structural design concepts.

RECOMMENDED RESEARCH FRAMEWORK

The purpose of this section is to present a research framework that can be used to provide continuity for research and to ensure that the completion of specific research projects will contribute to the solution of pavement rehabilitation problems.

In preparation of a framework for research, it is recognized that several formats could be used. For example, research objectives could provide one such framework. Specific objectives could be defined such as (a) development of overlay design procedure, (b) procedures for restoration without overlay, and (c) development of optimization models. A detailed enumeration of the various elements needed for each objective indicates that a considerable amount of overlap occurs in the individual elements. Although some overlapping may be desirable and even necessary, it tends to complicate the establishment of a research framework along these lines.

An alternative format for a research framework uses the concept of a flow network or process diagram. This format is built on the step-by-step process that can be used by an agency to develop programs for pavement maintenance and rehabilitation. This research format would be compatible with the process diagram prepared by Hudson and Finn (1) in Fig. 10 of their paper. The format also is similar in concept to that used by FHWA in preparing an outline for project V-D, Structural Rehabilitation of Pavement Systems. This alternative format was used to develop the proposed research framework.

A flow diagram illustrating the elements of the research framework is shown in Fig. 1. It illustrates the evaluation, design and decision processes. These processes will be discussed in general terms in the following sections of this report and then reduced to individual items including research needs.

Pavement Rehabilitation Process

Nine major activities associated with a rehabilitation process are indicated in Fig. 1. The purpose of network monitoring is to provide basic information about past and present pavement performance and to accumulate information useful in making a decision regarding rehabilitation. The two major requirements for information collected through network monitoring are relevancy, and reliability.

Careful consideration must be given to the factors that are relevant to the need for rehabilitation. The definition of performance must be resolved. Most engineers consider riding score and physical defects measures of pavement performance. The AASHO Road Test staff suggested that performance could be represented by a summary of the ride score (PSI) history. Haas, in his state-of-the-art paper, suggested considering the ride score as a user factor to indicate when rehabilitation is required and considering physical defects from the manager's (engineer) point of view to suggest the rehabilitation strategy (2). Performance, for certain purposes, can be defined as physical distress (thermal cracking, reflection cracking, faulting) or as riding quality.

Some agencies have combined the ride score with a physical defect score to produce a single statistic that can be used to measure overall performance and to predict when rehabilitation will be required. There is some advantage in a summary statistic, specifically in establishing priorities for rehabilitation if it is desired to consider both factors in making a determination for rehabilitation. The method has a disadvantage in that no indication can be obtained on the type of rehabilitation needed. The combined statistic gives no reliable indication of the real condition of the pavement.

Once performance is defined, it will be necessary to identify those factors that can be shown, through research or experience, to influence performance. Monitoring requires that some effort be made to measure both the performance parameter and as many of the influencing factors as possible. The accumulation of such information will help in the development of pertinent performance prediction models.

Reliability in the measurement is crucial to the development of prediction models. Monitoring (observations, measurements) should be accomplished with speed; however, data also should be accumulated in a systematic procedure that, in some way, includes a system of quality control on the measurements involved. For example, calibration, training, replication, and other factors should be considered in regulating the measurement program.

Analysis of the data (information) obtained from network monitoring is considered the next step in the rehabilitation process. The analysis would have two objectives: to estimate the levels of performance, both present and future, and to make preliminary evaluations of the costs, assuming that no rehabilitation is scheduled or implemented.

The first decision step in the rehabilitation process comes after the analysis has been completed. This decision is based on information obtained in the network monitoring phase. Comparisons with decision criteria indicate whether rehabilitation is required. If rehabilitation is not required, the specific project is returned to the routine of network monitoring; if rehabilitation is indicated, it is necessary to implement specific and detailed procedures designed to determine the optimum rehabilitation strategy.

When a decision is made that rehabilitation may be required, it is necessary to proceed with a detailed study described as a diagnostic investigation. This investigation provides the objective information needed for implementation of design strategies. Again, measurements and observations should be relevant and reliable as in the case of the network monitoring.

A preliminary analysis of information obtained from the diagnostic investigation will identify those types of rehabilitation required for the particular conditions observed or measured. For example, a low coefficient of friction will indicate the need for corrective action to restore skid resistance; pumping will indicate a need for base restoration and probably drainage improvements. Special problems will be isolated by the preliminary analysis including such items as low temperature cracking, the potential for reflection cracking, or special construction problems.

The output of information obtained from the diagnostic investigation and preliminary analysis will be the input to the development of design alternatives. The alternative design strategies generally can be classified into two categories; restoration and overlay. Overlays would normally be considered as the application of an additional layer of surfacing material to increase the structural capability (traffic carrying capacity) of a pavement.

Overlays also improve the riding quality and skid number. It is pertinent to note that overlays can be justified for purposes of preventive maintenance. For example, if the nature of traffic distribution is known to be significantly heavier (greater percentage of trucks or more weight per axle) than anticipated by design, an overlay could be justified. Some overlays may not improve the structural capacity of a pavement, for example, thin overlays designed to correct surface friction problems. Techniques to measure in situ strength properties are needed for estimating the structural effects of overlays.

Restoration includes a relatively broad range of activities such as:

1. Patching,
2. Base stabilization or restoration,
3. Replacement (localized areas),
4. Thin overlays for correction of surface disintegration or skid resistance,
5. Surface texturing, and
6. Surface sealants.

The output from the development of alternate design strategies will provide an array of rehabilitation alternatives. The way in which these alternatives are presented will depend on the optimization strategy, i.e., cost, benefits, safety, comfort. The selection of a specific rehabilitation strategy will depend on the optimization methods used to compare alternatives. At this point, the engineer-manager's decisions may interface with those of the political manager or the socio political manager, or other outside influences that may affect the final decision. The engineer's responsibility is to provide reliable, consistent, and objective information. The optimization models should include all the factors that the engineer believes, by either experience or training, to be important to the users of the facility as well as to the community of nonusers who may be affected by the decision. It should be remembered that one decision included in the rehabilitation strategy may be to do nothing, that is, to continue with routine maintenance.

Implementation of the selected rehabilitation will involve a number of important decisions. Construction techniques must be selected or developed. Traffic handling and controls must be planned, designed, and used in coordination with material selection and handling and equipment utilization. Environmental considerations must be included in all decisions.

Finally, the data bank system will complete the various phases of the total rehabilitation strategy. The data accumulation that supplements the network monitoring includes information related to rehabilitated pavements; measurements, cost and quality of the rehabilitation; performance after rehabilitation; cost of routine maintenance; and comparisons with design expectations.

In summary, there are nine steps involved in the rehabilitation process. It is hypothesized that, if satisfactory procedures are available or can be developed through research for each step, it will be possible for the engineer to provide a reliable, objective, effective pavement rehabilitation program.

The work flow diagram is offered as a general process to which research needs can be associated. Other investigators have suggested comprehensive systems diagrams that could be or have been developed for application to rehabilitation. The authors believe that research needs would be very similar regardless of the precise framework to be used.

Elements of Pavement Rehabilitation Process

In this section each step in the process will be evaluated in an attempt to enumerate research needs with particular emphasis on those items referred to by the various workshop groups.

Network Monitoring

Condition Survey -- The purpose of the condition survey is to describe the condition of the pavement at the time the survey is accomplished. By combining results from a series of surveys, we can determine the rate of change in the various factors evaluated.

Group I of the Workshop defined the condition survey "as any process of identifying, either qualitatively and/or quantitatively, visible manifestations of pavement distress." Thus, the working group did not consider riding quality within its specific responsibility. Because the Workshop was concerned primarily with structural rehabilitation, friction factors were not specifically enumerated although this condition should be considered for a complete condition survey. In the condition survey, the following three factors should be measured: distress (visible manifestations), longitudinal profile, and coefficient of friction.

Techniques for measuring distress, at the present time, tend to vary from agency to agency and depend on subjective estimates of the pavement condition. Six important problems were delineated by the working group on condition surveys:

1. Subjectivity in making survey,
2. Absence of valid sampling procedures for highway surveys (apparently the Navy has developed what it considers to be acceptable sampling procedures for airfield pavements),
3. Procedures for replicate surveys,
4. Lack of uniformity in weighting techniques,
5. Inability to store information for workable inventory, and
6. Hazards to personnel and disruptions to traffic.

Research needs identified by the working group were uniformity in techniques and mechanistic and objective procedures to improve the reliability of the condition survey. The major objective would be to produce information that was related to performance (by whatever definition of performance is required) and that would provide comparisons between projects for a given period of time and between intervals of time for a given project. Thus, methods of training are important, but methods to calibrate both equipment and personnel are especially important.

The third research need included in the working group's recommendations calls for investigations to develop the relationship between distress and performance. Research is needed so that weighting functions can be developed to establish the relative importance of various modes of distress, for example, the relationships, in importance, among cracking, rutting, faulting, and surface wear. Weighting functions are important for use as the dependent variables for prediction models and for proper establishment of decision criteria. It is pertinent to note that the principal research need reported at the Workshop was to relate distress to performance (3).

The research needs for longitudinal profile measurements would generally fall into the same categories as described for observations of distress: uniformity and objectivity. Also, the ability to relate longitudinal profile to performance or to distress should provide a complete framework for evaluation of the pavement's condition.

Measurements System -- There are two aspects to be considered in measurement systems: (a) what kind of measurements to make and (b) how to make the measurements. With each of these items it is necessary to consider whether the measurements are part of the network monitoring or of a diagnostic investigation. The thoughts of the working group on measurements programs were summarized earlier in this report. At this point the discussion is restricted to network monitoring.

The objective of the measurements program is to obtain information for a quick look at the pavements in the network. Specific types of measurements needed are longitudinal profile, physical distress, in situ strength, and coefficient of friction.

It is beyond the scope of this report to enumerate specific kinds of measurements necessary to obtain sufficient information for monitoring purposes; that would be the goal of a research project. However, it should be noted that a number of devices have been developed for measuring the longitudinal characteristics of a pavement (4). The working group has suggested that deflection measurements and possibly curvature measurements would be adequate for in situ strength evaluations. The coefficient of friction has been investigated, and methods should be available for its measurement. Objective evaluation of physical distress appears to be in need of further research to improve on current procedures.

Standardization of each of the methods of measurement will be extremely important, both for comparisons within an agency and for agency to agency comparisons. A good example of the absence of standardization is apparent in deflection research. Not only are there a wide range of devices, with their individual characteristics influencing the measurements, but for a given type of measurement there are a range of test procedures. For example, pavement deflection measurements can be made by using a Benkelman beam by several different procedures, e.g., the method used on the WASHO and AASHO Road Tests or as measured by the CGRA procedure. The measured values will be influenced by the procedure and so will the numerical criteria and interpretation.

The matter of how measurements should be made will require at least four considerations: speed, reliability, methods of sampling, and cost.

The working group has emphasized the need for speed in the measurements program. Any research investigation should also consider cost, i.e., the use of resources (time, equipment, personnel). It is recommended that monitoring include the maintenance of cost records for all phases of the process in order that benefit-cost studies can be made.

Confidence limits for the measurement program need to be established as an additional constraint. The scope of the network monitoring also must be considered.

Maintenance and Operating Costs -- One of the criteria suggested for the determination of the need for rehabilitation is the alternative cost of routine maintenance, the do-nothing alternative. For such criteria to be applied, it is necessary to establish procedures from which maintenance costs can be obtained and to evaluate maintenance costs in terms of value received (performance). Research on how to evaluate maintenance costs is important.

User costs should be considered in evaluating any rehabilitation strategy, and, therefore, information relative to user costs as affected by pavement rehabilitation would appear to be a significant research need.

Accident records also should be obtained so that such information can play a role, possibly in terms of cost, in the decision on rehabilitation. Also, long-term analyses should be directed to determining meaningful relationships among accidents, the quality of maintenance, and the skid number.

Operational and Environmental Conditions -- Research is needed to determine what operational and environmental conditions should be monitored to properly assess their influence on pavement rehabilitation programs. Consideration needs to be given to the standardization of pertinent parameters to describe the following:

1. Traffic -- For structural evaluations, equivalent axle load applications, average daily truck traffic, or aircraft count may be adequate. However, for coefficient of friction the total traffic volume will be most appropriate.
2. Climate -- Relevant rainfall and temperature parameters need to be determined. For example, is total rainfall useful information in determining the need for rehabilitation? Is average temperature useful, or would the total number of days below 32 F be more helpful?
3. Geography -- Does the type of terrain influence rehabilitation? If so, what descriptors should be recorded to describe different conditions in the field?
4. Noise -- Should or can observations relative to potential noise problems be included in network monitoring?
5. Roadside hazards -- Signs and obstructions could be included in field observations. Are such observations pertinent to rehabilitation?
6. Cultural environment -- What are the surroundings contiguous to the highway or runway? Which, if any, would be pertinent to pavement rehabilitation?

Summary -- These sections have attempted to enumerate those factors relevant to network monitoring of pavements within any specific system. It is apparent that large volumes of data could be collected in the network monitoring phase. Hence, considerable study will be required to determine what and how much information is needed. One of the major criteria in the determination of what is needed will depend on how performance is defined.

Data Analysis -- The network monitoring phase deals with the collection of data. The rehabilitation process must define those types of analyses required for a preliminary decision regarding rehabilitation.

Logically, all of the information from the network monitoring phase should be pertinent or useful to the analysis phase. A research need would, therefore, be to establish the methods for interfacing data with analysis, If there is no continuity then the data need not be accumulated.

Currently, three types of data analyses are required by the rehabilitation process: (a) performance predictions, (b) cost predictions and summaries, and (c) environmental impact and the socio-political conditions of the area.

Performance Predictions -- It is not the purpose of this report to define performance. However, it is obvious that performance must be defined before it can be predicted. Both performance and performance predictions were discussed at the Workshop.(1,2). Virtually all of the working groups have mentioned the need for prediction models. This represents a prime research need. The inputs of the prediction model would be obtained from the condition survey.

The purpose of the performance prediction model will be to estimate the condition (e.g., PSI, physical distress, coefficient of friction) of the pavement at some future time. It will be necessary to consider the decision criteria to be applied; i.e., what types of constraints or limiting values on the various types of performance are to be required by management. One other objective of the prediction model will be to define confidence intervals for predictions, in terms of both time and condition.

Cost Predictions -- The objective of a cost prediction model is to predict the cost of doing or not doing rehabilitation. Costs will be the total of routine maintenance and user costs. The development of cost models is a major research need because a minimum of effort has been expended in this area to date.

Cost predictions will also need to include information relative to the cost of maintenance after rehabilitation is completed. Such factors as the thickness of overlay and construction procedures will influence performance and cost.

Preliminary Decision -- Based on the network monitoring and analysis, an initial comparison with pertinent criteria can be made. The purpose of this initial comparison is to determine whether projects require rehabilitation.

To make this initial comparison requires that a series be established of decision criteria. Just what these criteria might be is a research need at the present time. It is suggested that criteria could relate to (a) performance standards, (b) cost limitations, (c) safety, (d) environmental impact of not correcting certain conditions, e.g., noise, and (e) socioeconomic considerations, e.g., rehabilitation could increase traffic flow, which has both social and economic impacts on the area.

The alternative decision is to do no rehabilitation, in which case the project is returned to the network monitoring population where it will be routinely observed at prescribed intervals or to recommend further evaluations. The latter category will include pavements for which some kind of rehabilitation is considered mandatory, e.g., low coefficient of friction.

Diagnostic Investigation -- The purpose of the diagnostic investigation is to obtain detailed information necessary for an evaluation of various rehabilitation strategies. In some ways, this investigation will be similar to the network monitoring. Condition survey information and physical measurements will be required. However, the exact nature of the survey and measurements will be significantly revised as compared to network monitoring.

Condition Survey -- The need for objective and mechanistic methods to obtain condition survey information will become increasingly important for the diagnostic investigation. Also, a significant difference will exist regarding survey objectives. In the network monitoring, primary interest is centered around the types of distress and their rate of progression. For diagnostic purposes it will be important to determine the cause of distress and to obtain basic information for use in determining alternative rehabilitation design strategies.

One of the major objectives of the condition survey will be to determine whether special problems, such as low-temperature cracking or poor drainage, are present that may require special treatment.

The working group on reflection cracking has suggested that an important research need is to develop guidelines for the collection and analysis of data pertinent to cause and control of such cracking.

Measurements Program -- The measurements program, like the condition survey, will need to acquire such information as will be required for the development of rehabilitation design strategy. The working group has enumerated a range of items that should be measured at this stage, e.g., deflection, curvature, modulus. Other factors are pavement thickness, pavement profile, skid number, and environmental conditions.

A research need is to identify what needs to be measured, how and with what accuracy and precision.

Data Analysis -- This step represents an initial evaluation of the information obtained in the diagnostic investigation. It serves to better identify the problem and to enumerate the types of rehabilitation strategies to be included in the next phase. The question asked of this evaluation is what types of rehabilitation can be considered at this stage? For example, for urban freeways, will user costs eliminate certain types of rehabilitation? The research need is to define criteria that can be used to select the most likely types of rehabilitation suited for a particular project.

Design Alternatives -- For purposes of this report design alternatives are divided between overlay and restoration, including replacement of limited areas.

Overlay Design -- The major approach to rehabilitation is the use of overlays. The thickness of the overlay, the materials used, and the construction procedures are each relevant to the determination of the overlay requirements.

Witczak (5) has summarized overlay design procedures for airfields and McComb (6) for highways. The procedures described have been categorized into two types: component analysis and in situ strength. There are similarities in each; however, the differences are enough to justify the division.

The component analysis involves three basic steps: (a) measuring the properties of materials, (b) measuring the condition of the existing pavement, and (c) comparing the evaluated thickness of the existing pavement to that of a new pavement designed for the same conditions.

The methods for evaluating the strength of an existing pavement were described by Witczak (5) and include the possible use of the layered elastic analysis as well as more conventional methods such as k-value, CBR, or R-value.

Methodology for the component analysis procedure is generally available at the present time. Some research is needed, as suggested by the working group, to develop reliable methods to estimate remaining life by such procedures.

The in situ strength procedures require a measurements program that will allow the total system to be evaluated in place. With this capability, there would be no need to develop laboratory procedures to estimate the in-place condition.

The working groups have indicated that research relative to deflection, deflection basins, curvature, and wave propagation is needed. Also, the influence of seasonal variations on in situ measurements needs further evaluation.

Research is needed to establish design criteria for overlays by either of the above methods. Research needs to consider design criteria in terms of materials and construction techniques. For example, overlay requirements could be influenced by efforts to strengthen the subsurface layers or by improving the uniformity of support of the existing pavement structure as pointed out by the working group on materials and techniques.

The use of experience in highway engineering has been the basis on which most overlays are designed. In all probability this procedure will continue to be one of the principal ways in which an overlay is determined. One of the major research needs is to systematically catalog the information so that this experience can be stored and evaluated. Thus, the working group on other procedures has suggested the need for an appropriate record system for control of overlay designs, construction, and evaluation of results.

Restoration -- The technique of restoring an existing pavement, without an overlay, offers an important rehabilitation alternative. Research needs were discussed by the working group on materials and techniques. The general categories of rehabilitation by this technique would include base restoration, patching with precast sections, subsurface injections, leveling (paving, planing), slab replacement, and, possibly, application of surface treatments to seal and rejuvenate. Questions on when to use such techniques would include the following:

1. How inconvenient would it be to the traveling public?
2. How effective is the procedure?
3. What is the cost?

The establishment of working procedures is an important research need for rehabilitation without overlays.

Selection of Rehabilitation Procedure -- Based on the implementation of alternative strategies, it should be possible to effect an array of choices in terms of a specified optimization procedure. The working group on strategies has offered a number of alternatives as a basis for the development of optimization criteria. These have been covered by McCullough and Epps and in the first section of the Workshop report.

It is significant that research needs described by the working group included, in addition to cost, factors such as safety, comfort, sociopolitical impact, environmental impact, benefits, and available resources.

On the basis of the array of choices provided by the optimization phase, a decision on rehabilitation can be made. One such decision would be to continue with routine maintenance, the do-nothing alternative. In the event that the do-nothing alternative is selected, project data would be returned to storage and routine network monitoring would be continued.

Implementation -- Research relative to the implementation phase would involve the following:

1. Project planning,
2. Traffic control,
3. Equipment utilization,
4. Materials handling, and
5. Environmental protection (noise, air, and water pollution).

Whereas these items are not necessarily part of the design determinations, they represent for rehabilitation an important aspect of a successful project. Byrd (7) has discussed aspects of traffic handling; the working group on materials and techniques has discussed construction techniques.

Data Bank

Most working groups indicated that suitable data banks need to be established as a means to store and retrieve pavement information. The working group on other procedures indicated that one research need is to "define and design a data bank model to incorporate necessary elements of overlay design, evaluation, and control plus compatibility with total needs of a pavement management system, including ready retrieval capabilities."

The purpose of the data bank is to accumulate information needed to evaluate the effectiveness of the selected rehabilitation strategy and to develop future design criteria. Factors relating to performance, design criteria, and the influence of material properties and construction on performance all need to be determined, measured, stored, and evaluated. A research need is to establish precisely what factors need to be measured and how they can be used as feedback to the rehabilitation process.

SUMMARY OF RESEARCH NEEDS

Earlier in this report an effort was made to summarize specific research needs as recommended by the working groups. Then a framework was described for the rehabilitation selection process. The framework illustrates how various research needs ultimately can provide the information required by the engineer-manager in making decisions relative to pavement rehabilitation.

This section summarizes research needs, specific research projects by priorities, and research needs within a work flow diagram for rehabilitation.

Table 1 gives research needs in general categories according to the availability of past research. Second order research needs requiring additional research and development are as follows:

1. Develop highspeed measuring equipment for evaluating in situ condition of pavement including such factors as physical condition, remaining life, and skid number.
2. Develop mechanistic procedures for predicting (controlling) reflection cracking.
3. Develop highly flexible materials that can be used as thin overlays.
4. Develop mechanistic procedures for the determination of the state of stress or strain in a layered system with cracks in the upper layer.
5. Obtain and evaluate information pertinent to establishing the inherent variability associated with specific material properties, construction procedures and performance predictions.

Figure 1. Research framework for pavement rehabilitation.

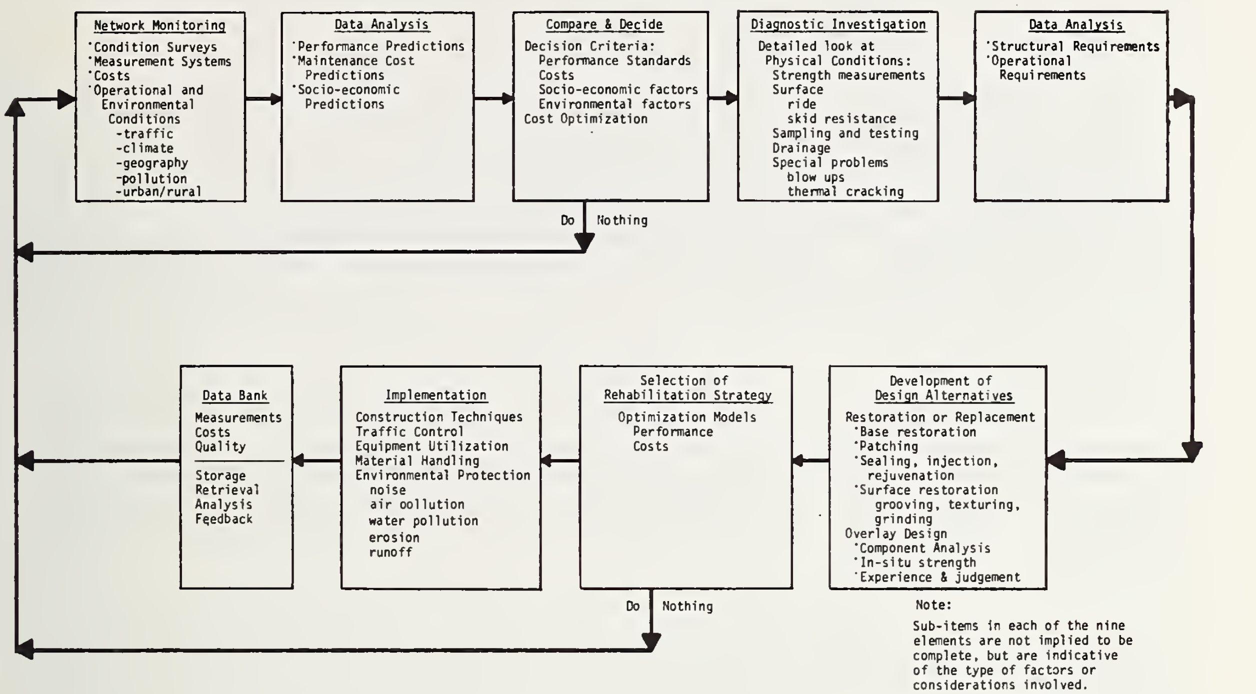

Table 1. General research needs based on available past research.

Status	Guidelines
Implementable	Standardize methods for evaluating condition of pavements Obtain relevant cost information for maintenance and rehabilitation Develop performance prediction models for pavements before and after major maintenance or rehabilitation Describe a data feedback system relevant to pavements with and without major maintenance or rehabilitation Develop data bank systems for storing and retrieving information pertinent to pavement management systems Develop pavement management systems to optimize rehabilitation decisions
Requires additional research and development	Develop new materials specifically designed for use in major maintenance or rehabilitation that will minimize the inconvenience to the traveling public and that will · minimize air and water polution · minimize energy requirements · minimize the thickness of overlays · maximize the use of on-site materials Develop standardized design procedures to determine overlay requirements for various materials by using some combination of objective measurements and mechanistic and probabilistic theory

Table 2. Guidelines for research categories.

Category	Guidelines
Implementable	Research needs that can be satisfied by a comprehensive synthesis of available research findings into engineering criteria, specifications, and data for application to rehabilitation problems
Requires additional research and development	Research needs for which available information is not complete and for which new development will be required before implementation is possible
First order	Research needs required to provide basic engineering criteria or procedures necessary to satisfy a rehabilitation process such as that described in Figure 1
Second order	Research needs that recognize weaknesses in available procedures and are required to strengthen, modify, or develop better techniques

Table 1 and the list above are incomplete but represent a method of assignment of research needs on the basis of current research findings. Assignments into research categories were made according to the guidelines given in Table 2.

If research needs can be organized into categories as suggested, it is possible to assign funding priorities in the order of probable payoff as follows: (a) implementable research in first order category, (b) research and development in first order category, and (c) research and development in second order category.

It is important to recognize that assignment of research needs into categories as illustrated by Tables 1 and 2 will require a comprehensive evaluation of the problem and the availability of current technology (research) responsive to that need.

It is not intended to suggest that research needs in second or third categories be considered as less important. The categories merely suggest an order in terms of time for payoff. Completely satisfactory procedures for pavement rehabilitation will not be obtained until research needs are satisfied for each category.

CONCLUSIONS

The satisfaction of the research needs described in this report will require cooperation and coordination on a national and international scale in order for the results to be realized at minimum cost and with maximum usefulness to those agencies charged with the responsibility for pavement rehabilitation programs. These agencies must recognize that research and implementation can go hand in hand even though the development of an ideal world rehabilitation program will be a step by step process that can encompass a considerable amount of time and verification.

Where field or laboratory data are not available to initiate research it may prove expedient and effective to use the experiences of practicing, experienced engineers as an initial source of data and information. When data feedback systems begin to function, the information based on experience can be combined with measurements to strengthen the data base. It is important to have a clear understanding and definition of pavement performance and to establish a relationship between performance and distress. This is a high-priority need for both pavement rehabilitation and pavement design management systems.

The benefits to be obtained by satisfying the research needs described are considerable. Benefits can be measured in terms of (a) more efficient and effective use of available funds, (b) more objective and consistent decisions regarding the accomplishment of rehabilitation, (c) better use of better materials (conservation), (d) ability to optimize rehabilitation decisions according to cost, energy, use of materials, social impact, and so on, and (e) providing the engineer-manager with the reliable tools necessary to manage a pavement network of highways, runways, taxiways, or other paved facilities.

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EVALUATION OF THE PAVEMENT REHABILITATION WORKSHOP

B. Frank McCullough and Jon A. Epps

In recent years, the need for a more rational design procedure for rehabilitating existing highway and airport pavement structures has become more pronounced. Immediate needs always serve as a catalyst for improvement; therefore, as engineers are faced with, for example, rehabilitating major highways built in the early part of the Interstate program or rehabilitating existing runways to take care of a new series of aircraft, the need for a more rational rehabilitation design procedure becomes critical. Thus, through a continuing effort of many individuals and agencies, the Workshop on Pavement Rehabilitation was conducted by the Transportation Research Board through sponsorship of the Federal Highway Administration.

The overall objective of the Workshop was to establish a better understanding of pavement rehabilitation concepts, operations, and needs so that the framework for a rational pavement rehabilitation design procedure could be formulated and used as a guide for future research and development studies. Thus, the limited national resources available to "tackle" this problem could be optimally expended, since all agencies would have a common set of goals. The Workshop and subsequent papers and reports developed a documentation that would serve as a vehicle to accomplish these objectives by providing a compendium of information and direction developed by a cross section of competent individuals working in this area.

NATURE OF WORKSHOP

The 4-day Workshop consisted of nine sessions. The mode of operation called for two basic session types: (a) state of the art or "catalyst" sessions and (b) group meetings or "brainstorming" sessions. The session types were selectively dispersed through the program.

The "catalyst" sessions were preplanned by assigning experts to prepare the following papers covering topics in planning principles, operating techniques and procedures, and special problems:

A. Planning principles
   1. A General Framework for Pavement Rehabilitation by W.R. Hudson and F. N. Finn.
   2. Pavement Rehabilitation: Highway Maintenance Problems by Travis Smith.
   3. Urban Area Problems Associated With Pavement Rehabilitation by L. G. Byrd.

B. Operating techniques and procedures
   1. Surface Evaluations of Pavements: State of the Art by Ralph Haas
   2. Drainage of Pavement Rehabilitation by George W. Ring
   3. Structural Evaluation in Overlayed Design for Highway Pavements by R. J. McComb and J. J. Labra.
   4. A Structural Evaluation and Overlayed Design Methodology for Airport Pavement: State of the Art by M. W. Witczak.

C. Special problems
   1. Special Problems With Air Force Maintenance by D. I. Hanson.
   2. The Thermal Cracking Problem and Pavement Rehabilitation by W, A. Phang and K. O. Anderson.
   3. Reflection Cracking by George B. Sherman.

The "brainstorming" sessions were organized by dividing the attendees into working groups where there could be a more informal flow and interchange of ideas. A chairman presided over each group and was responsible together with the group recorder for preparing the group reports. Two sessions a day apart were held by each group to provide for a preliminary interchange session, a reflection period, and a summary session. Groups with special topics for discussion were divided into two basic categories:

Category 1: Rehabilitation techniques, strategies, and problems
1. Condition surveys
2. Measurement systems
3. Materials and techniques

4. Strategies
5. Reflection cracking
6. Other special problems

Category 2: Thickness selection procedures

A. Modification of structural design procedures
B. Layered elastic systems procedures
C. Deflection curvature and stiffness based procedures
D. Other procedures
E. Ideal procedures

The authors attended each of the catalyst and group sessions to feel the pulse and general trend of discussion. In addition, the draft reports of each group and the catalyst papers were made available to the authors for study and evaluation.

The objective of this paper is to evaluate the workshop proceedings including the state-of-the-art reports, group deliberations, and group reports. This report appraises the current state of the art of pavement rehabilitation and provides value judgments concerning what is known, what is implementable, and what is not known.

After the workshop, the authors were faced with a wealth of information to summarize. Because the summary could be organized in numerous ways, it was felt that the most appropriate format to follow was the general outline established in Fig. 1, which was selected from the paper by Hudson and Finn (1). Thus the remainder of this report is arranged as follows: fundamental concepts for pavement rehabilitation, inputs, models, behavior, distress, performance, and costs and special problems.

One product of each group session was the preparation of numerous tables defining various areas of pavement rehabilitation. The authors have attempted to consolidate this information in a series of tables relating to each section of the report. The tables are illustrative and are not intended to be all-inclusive. The authors have attempted to resolve areas of conflict by considering the overall direction and emphasis of the workshop.

The workshop dealt with rehabilitation of both airport and highway pavements. It may seem that the material is oriented toward highways, but this may be attributed to the fact that much of the previous development was in the highway field. The concepts are generally applicable to both airports and highways. In general the approach used herein is that a given topic is generally applicable to both pavement types unless specifically stated otherwise.

FUNDAMENTAL CONCEPTS FOR PAVEMENT REHABILITATION

The conference participants represented a variety of backgrounds and responsibilities; thus numerous ideological approaches were injected into the meetings. In the first section the general ideology present in the sessions are summarized into what is "hoped for but not yet achieved" (ideal world) and "what is immediately needed and is achievable" (present world). The last two sections deal with implementation and research needs.

Ideological Approaches

Hudson and Finn (1) in the opening session of the conference stated that historically pavement design has been thought of as a "one-shot" process that could be done with complete confidence if we had rational methods. The trend of the conference indicated that the participants were not thinking in this direction, but rather recognized it as an iterative process. In the papers and group sessions, two fundamental ideological approaches were evident although not necessarily recognized nor defined by the participants.

One general approach comes from the practitioner, faced with the pressures of current pavement design problems, expressing his desire for procedures and techniques for correcting immediate problems. The objective is to make better use of available materials, manpower, and equipment. The second approach comes from those wishing to do a better job of optimizing resources (i.e., available money, time, and materials), calling for a reevaluation of the past methods used in solving the problem. In essence, this group is seeking to broaden the design approach. These statements do not imply that there is a clean line of division between "individuals" and "camps of thought." Generally, an individual may subscribe to one approach in one case where his personal interests or activities are involved and subscribe to the other when he could be in a more objective frame of mind.

Numerous examples of each ideology were evident in the meetings. For example, the Group D report (2) enumerates the major problems facing the practitioner:

1. The need for a more economical method for maintaining skid resistance on portland cement concrete pavements throughout the structural life of the facility, and
2. A need for a more economical means of rehabilitating structurally distressed travel lanes of multilane roadways where the passing lanes show no evidence of distress.

Another practitioner approach comes from the Group 4 report (3), in which the specific areas of concern to the engineer are (a) rehabilitation of thermally cracked pavements, (b) rehabilitation of high-volume and high-speed pavement, and (c) rehabilitation of pavements in view of environmental and ecology conscious individuals.

The systems engineer's desire for a more objective viewpoint is reflected by Smith's statement (4).

> "It would be highly desirable to have an information system developed that would predict when maintenance was needed on a section of roadway, when work was needed, how it should be performed, and how much it would cost."

The Group E committee (5) expressed concern for a universal approach encompassing all the design parameters needed for various geographical areas. Thus the development should not be toward developing a standard model that does not recognize these sensitivities, but toward a universal model.

It is evident that both groups are attempting to solve the same problem, but the mode of operation and the desires are different. There are relative merits in both approaches and possibly with proper planning the objectives of both approaches may be achieved.

Ideal World

Those subscribing to the systems approach are seeking to recognize that pavement rehabilitation design is actually an iterative decision-making process for developing models or techniques whereby resources are optimally converted into desired ends. People sometimes think systems engineering implies sophisticated computer programs and models. It cannot be stated too frequently that economic choice is a way of looking at problems that does not necessarily depend on the use of analytical aids or computational devices. Some mathematical models and computing machinery are quite likely to be useful to arrange alternatives, but they are not a necessity. Where mathematical models are available, they are in no sense alternatives to or rivals of good intuitive judgment; instead they complement it. Judgment is always of prime importance in selecting the criterion for design and the proper alternative.

In a systems approach, the logical steps are first to evaluate the existing pavement structure and then to develop decision criteria pertaining to the evaluation. The next step is to consider the possible rehabilitation activities and formulate the various strategies that may be used. These various strategies should be optimized in terms of cost and a final design strategy selected using the pavement feedback data system.

Evaluation -- Logically, the first step is to evaluate the existing pavement. Haas (6) has stated that the evaluation of existing pavements for rehabilitation needs can be done in terms of

1. Structural capacity,
2. Physical deterioration,
3. User-related effects such as roughness, safety, and appearances, and
4. User-related cost and benefits.

Decision Criteria -- After evaluation the pavement, the next logical step is to consider the various reasons for pavement rehabilitation such as outlined by Hass (6) and the Group 4 report (7)

1. Inadequate structural capacity,
2. Unacceptable level of serviceability,
3. Unacceptable level of distress,

4. Unacceptable level of safety,
5. Unacceptable user cost, and
6. Unacceptable routine maintenance cost.

When these reasons are quantified in terms of limiting values, the decision criteria are then formulated for use by the designer, A major factor in applying the decision criteria may be influenced by current or future plans concerning the function of the facility as indicated by Hudson and Finn (1).

Rehabilitation Activities -- The designer must next consider the various rehabilitation techniques available to him as reported by Group 4 (7) and to some extent by Smith (4).

1. Overlays;
2. Surface rejuvenation through rejuvenators and/or seal coats;
3. Major maintenance activities;
4. Subgrade, subbase, and base strengthening through injections, stabilization, drainage, and so forth;
5. Reconstruction, i.e., removal and replacement;
6. Manipulation and reworking of in-place materials;
7. Application of materials to pavement surfaces to strengthen the surface materials;
8. Application of erodable surfacing;
9. Grooving of pavement surfaces to improve skid resistance; and
10. Various methods of protection of pavement components from the elements such as soil encapsulation and insulating layers.

Strategies and Optimization -- Each of these techniques provides a possible solution to the problem. These become the alternative strategies that must be evaluated. Because several types of deficiencies may be present, the Group 2 and Group 4 (7, 8) emphasize that the action to be performed should be defined and objectively rated in a traditional investment nature. Group 4 (7) has recommended the use of a benefit-cost ratio for evaluating alternate strategies. Thus the engineer should seek to maximize the strategy's worth, which is the difference between the benefits and cost.

Hudson and Finn (1) take the evaluation one step further:

"To deal with rehabilitation strategies requires that the needs of both the highway network and a particular project within the network be considered."

Thus a comparison must be made of exclusive alternates, selecting one project or the other but not both. In each of these alternates, risk and uncertainty must be recognized and accounted for through stochastic techniques.

Feedback Data System -- Numerous committee sessions and formal papers recognize that the pavement structure must be continuously monitored and a feedback data system developed. This system can be used to upgrade the model in the design system and also to provide information for future design.

Present World

The present world of design reflects current practices and desired changes that are immediately feasible. In contrast to Haas' needs for rehabilitation in an idealistic framework, the Group D report (2) states that pavement rehabilitation needs are determined by structural improvement, restored rideability, and road safety. The decision criteria, for example, as per the Group D report, for determining when a pavement is to be rehabilitated are condition surveys and/or public or political pressure.

Most of the quantitative procedures associated with current rehabilitation involve structural design models. Because the bulk of this report pertains to the current state-of the art and structural design, it will be considered here only in a passing manner. The Group A report (9) includes a discussion on whether immediate structural model improvement should be modifications of existing procedures or development of so-called "virgin" procedures. The advantages of the modification approach were listed as follows:

Figure 1. Simplified predictive portion of pavement system analysis relating six methods of evaluation.

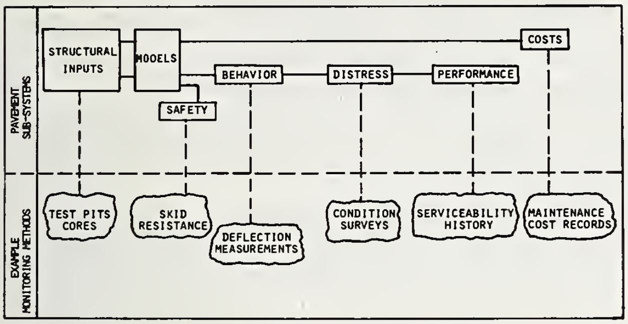
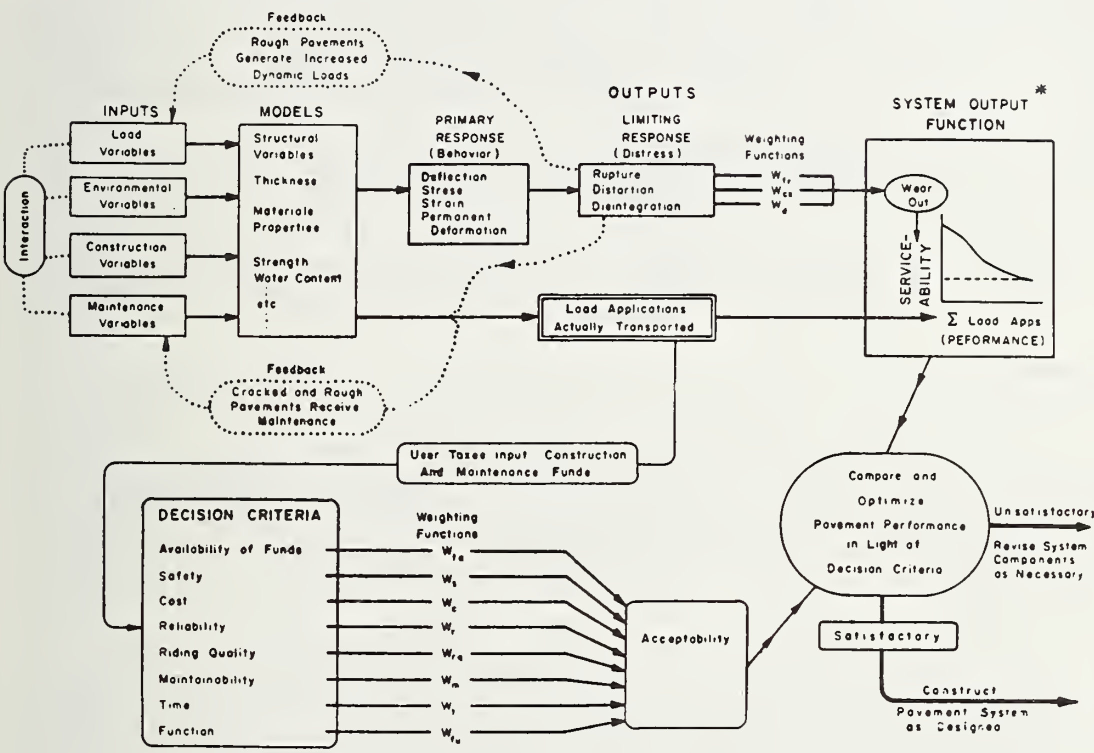


Figure 2. Conceptual pavement system (20).



1. The procedures are more easily implemented because less additional training is required,
2. Such an approach is compatible with existing pavement management systems, and
3. Inherent weaknesses in the original pavement design procedure may be exposed.

The disadvantages of the modification approach were given as follows:

1. Weaknesses such as basic assumption errors are perpetuated in the modification;
2. Existing procedures are generally empirically based and extrapolation of such procedures is questionable; and
3. Current procedures apparently foster the tendency not to evaluate objectively the in-place condition for the design of the overlay.

These lists, although aimed at structural models, reflect the philosophy of the present world approaches, which are limited in their overall objectivity. Obviously, many other rehabilitation techniques are probably considered subjectively by all designers but not in an organized format.

Implementation

In each of the following sections of the paper, implementation statements are made as to the material covered. Because most of this information follows the so-called present world format, emphasis is made here for need in implementing the ideal world concept. Certainly there is an immediate chance of payoff because currently a subjective evaluation of all the previously described rehabilitation techniques are made, but not in a systematic framework for checking the interactions of the decision criteria. This conference points to the need for establishing this broad framework so that future development and research work will have both a short and long range payoff.

With reference to the remainder of the report, a rating scale of 1 to 3 is used to list the current status of implementability. The scale is as follows:

Rating	Description
1	Considerable research required prior to implementation
2	Limited research prior to implementation
3	Immediately implementable

Research Needs

The first major step is to develop an ideal world rehabilitation design procedure. This framework could be used as the planning document for investing research and development money, available manpower, and materials in solving the present world problems. Thus, the immediate problem is solved, but in a manner that the immediate developments can be used in a more objective procedure. All investigators involved in this problem should recognize that research and implementation go hand in hand and that the development of an ideal world rehabilitation procedure will be a step-by-step activity spanning many years.

Inputs are quantified values of parameters or variables contained in the various models in the pavement system. Fig. 2 is a conceptual diagram of a pavement system illustrating the relationship between inputs and models. As can be seen, there are numerous classes of inputs. Each of the inputs must be characterized with the specific model for which it is to be used. In many cases, the same input value may be used in several models, whereas in other cases different characterization procedures may be required by different models. An example of this, in the materials characterization area, is the stiffness value of the asphalt concrete. In a structural model, the asphalt concrete stiffness may require characterization under a rapid loading cycle but with a slow loading cycle for the reflection cracking model. In contrast, Poisson's ratio obtained under any reasonable loading time may be adequate for either model.

An obvious deduction of the above is that the selection of input values must be closely related with the model. Thus, ensuing discussions will not be directed toward the specific inputs, but rather toward the classes of inputs.

Another type of input is the decision criteria or limiting values that place constraints on the system. Most of these input values are required in the optimization process. The decision criteria place the bounds on the analysis activities.

Existing Categories of Input

Although many of the papers and group reports touched on the categories of input, no single universal grouping was evident. For purposes of this paper, the inputs are divided into six basic categories of safety, pavement structure, cost, maintenance, traffic, and optimization of strategies.

Safety -- One of the inputs for the safety model is the skid number. Both Haas (6) and Hanson (10) describe the complexity of characterizing skid resistance. Generally, the parameter is defined as the skid number:

SN = f(velocity of test, water depth, type of tire, tire wear, vehicle maneuver, aggregate texture, pavement variables)

Most test procedures for the skid number attempt to standardize these parameters. Material properties other than the parameters covered by the testing procedures can have a major influence on the history of the skid number. Parameters such as surface texture and material hardness are inputs required to predict a skid number history such as shown in Fig. 3 by Haas (6).

Structure Inputs -- The structure inputs are numerous and range from layer thickness to complex material properties. The applicable concept derived from the papers and sessions for material characterization indicated that the testing should be performed in an environment as close to the in-place conditions of stress, temperature, and moisture as possible. For example, the Group B report (11) emphasizes that the inability of elastic layered theory to take into account the stress dependency of the modulus of elasticity for some materials was a major limitation in its implementation.

Cost -- Cost input ranges from the fairly well-defined value of material costs to an ambiguous user delay cost derived from assigning an hourly cost rate to individuals and vehicles delayed in traffic by maintenance operations. Obviously the reliability of these two data points differs substantially, but they may be used with equal prediction reliability in a model. In addition, inflation and present value computations influence the cost values assigned at various operational points during the facility's life. Although this was recognized to some extent in most of the sessions, the present state of the art gave only a cursory recognition to these factors.

Maintenance Inputs -- The maintenance inputs in current practices are primarily reflected in the cost category together with some consideration for the amount and type of patch.

Traffic Inputs -- Traffic inputs are generally in the form of equivalent wheel loads and average daily traffic (1,6). The equivalent wheel load may be derived by different methods, but the general concept is to convert mixed traffic applications into a design traffic number. Growth rates may be necessary in some of the models. In addition, computations of delay time may require alternate traffic flow patterns for rehabilitation operations.

Decision Criteria and Optimization Inputs -- One of the more important inputs is the length of the analysis period (7). The length of the analysis period will have a pronounced influence on the optimum design strategy. The level of service expected for a facility will also be a major factor in selecting the length of the analysis period. For example, shorter analysis periods may be expected for low-traffic rural areas, whereas longer analysis periods may be required for high-traffic urban areas.

Adequacy of Existing Inputs

The input characterization for the existing state-of-the-art models is probably adequate in most instances. As the models are improved, the characterization of the inputs will need to be altered and improved. One immediate area of desired improvement in the materials area is to ensure that the characterization is performed with stress, temperature and moisture conditions simulating field conditions and to verify laboratory values with actual field values (11).

Furthermore, most input parameters are random variables; thus, the models should be capable of taking into account input parameter variability. Unfortunately, this is not the case in the present state of the art, but provisions should be made for this in future developments.

Implementability

The implementation statement on input parameters depends on the implementation capability of the models. If models are adequate then the input values are implementable.

Current Research Needs

The primary research need of input parameters is to establish the variability of the values. The variability should take into account not only the standard deviation but also the type of distribution function that most appropriately describes its density.

Another essential area is acquiring better maintenance cost data on the breakdown relative to maintenance activities including types of operations and costs for travel, equipment, personnel, and safety. Also required is the effect of routine maintenance on the performance history of the pavement.

MODELS

A model is a system of postulates, data, and inferences presented as a mathematical description of a conceptual reality. The model provides the designer a mode of simulating the performance of a pavement structure without constructing a test section for each possible strategy. The Group 4 report (7) states that the development of an optimum strategy implies an ability to predict performance trends with reasonable accuracy and to predict the concequences of maintenance operations.

Discussions, presentations, and group reports during the Workshop directly and indirectly implied the advantages and disadvantages of using mathematical models. The obvious advantage is the rapidity with which numerous strategies can be derived and economically optimized. The major disadvantage is the lack of follow-up, which is required for checking the validity of assumptions invariably made in deriving the model. Hudson and Finn (1) recognize this point:

> ". . . a few agencies have attempted to obtain information required to establish and/or verify prediction models."

Figure 2 conceptually illustrated the relationship between models and inputs. Hudson and Finn (1) state that the predictive models must include three major parts:

1. Models related to skid resistance and safety = f(skid number),
2. Cost models = f(structural, maintenance, traffic, delay time), and
3. Performance prediction = f(traffic, structure, maintenance).

The combination of model categories 1 and 3 provides the design strategies. When these design strategies are compared with the cost models, a fourth category of predictive models is obtained by this optimization. Thus optimization is the function obtained by combining the three predictive models listed by Hudson and Finn.

No current procedure encompasses all the models discussed in the following sections, but they are considered directly or indirectly. The objective of an ideal procedure is to put these models into a common framework so that the interactions can be considered.

Existing Models

For each of the six model categories previously enumerated, numerous submodels were discussed or presented during the course of the workshop. Table 1 gives a summary of the submodels available in each category briefly describing the function of each, its advantages or disadvantages, and its implementability.

Safety -- One submodel of safety is the skid number, which predicts the decay in skid resistance with traffic and climatic effects. Most of these models are now based on existing roadway sections or extrapolations of test track data to field performance (6). There should be a continuing development for predicting the skid number as a function of material properties.

Smith (4) made reference to a safety index that would relate exposure of maintenance employees with different types of activities to the probability of an accident. He emphasizes that this may be as valid a criterion for a major rehabilitation as other factors normally considered such as riding quality.

Structural -- Considerable discussion and effort in the various sections were directed at models for predicting cracking in the pavement structure as is evident by submodels in Table 1. The structural number model covers other modes of distress, but they are primarily traffic initiated. The reader should recognize that the structural model may use more than one submodel, but generally only either the elastic, deflection, or structural number model will be used. The discussions in the Group C session (12) were directed to the lack of consideration for non-traffic-oriented distress manifestation and mechanisms such as settlement and swelling clay.

McComb (13), Witczak (14), and the Group B (11) express the feasibility of implementing elastic theory and the possibility of achieving roughness due to deformation as well as the cracking distress. The Group B report (11) recommended seven major factors to consider before applying elastic theory.

1. Applicability to new or nondistress pavements,
2. Applicability to cracked pavements,
3. Material characterization,
4. Failure criteria,
5. Remaining life concept,
6. Applicability to performance model, and
7. Current implementable procedures.

The major lack of knowledge is related to area number 4. Substantial effort and money should be expended on future studies directed toward developing a transformation relating computed parameters and areal distress.

The Group C report (3) examines 10 deflection based models and gives an excellent critique of each. Although there were a number of common threads between the various procedures, it was apparent to the group that none of the systems provided criteria for universal application. In general, none of the systems appeared to be interchangeable because of differences in their relationships of deflection to performance. The committee concluded that these differences must be attributed to the variations in materials, environment, and measurement techniques. The committee also stressed that caution should be used in attempting to adopt one of these procedures, especially in light of the previous discussion.

Both the reflection cracking model discussed in the Group 5 report (15) and by Sherman (16) and the thermal fracture models discussed by Phang and Anderson (17) and Group 6 (3) seek to predict stresses (cracking) in the pavement structure, which occur as a result of shrinkage forces from low temperature, The basic difference between the two is that reflection cracking has the compounding factor of an underlying layer (generally, a jointed portland cement concrete pavement) imposing stresses due to volumetric change on the overlay. The major limiting factor in each of these models is the development of transformation between stress prediction and areal cracking.

Cost -- The costs that must be considered include not only initial construction costs, but also (a) routine maintenance cost, (b) rehabilitation cost, and (c) so-called user cost. User cost are those costs that the pavement user pays, both directly and indirectly, in relation to the pavement facility or lack of it. These costs are primarily related to a pavement in poor condition, which results in excessive roughness, perhaps loss in speed, and vibration damage to the vehicle. A second major type of user cost is related to detour and delay costs that the user suffers in relation to time required for maintenance and rehabilitation of a given facility. Considering these costs along with the initial construction cost and the time value of money makes it possible to evaluate true relative cost of various pavement rehabilitation strategies.

Maintenance Models -- Maintenance models in the true form are not available. Most of the present consideration comes from cost models in which maintenance cost rates are used as an input. It is apparent that more information is needed relative to the effect of maintenance operations on the performance history. Furthermore, Ring (18) shows the importance of drainage on performance, pointing up a need for development work relating drainage characteristics and pavement performance.

Traffic Models -- The equivalent wheel load models have been used extensively for converting mixed traffic into one number that can be used for a structure model. Although there are several submodels available, they are all conceptually similar.

Byrd (19) discusses the need to apply traffic flow models and also provides examples of their effects. These models permit a relative comparison of traffic handling during rehabilitation operations. The time of delay data obtained from these models is translated into user cost in the cost models.

Optimization of Design Strategies -- The optimization process comes from comparing the costs of the various design strategies developed from the above models. This is a time-consuming process that requires the use of a computer to fully cover all of the possible strategies.

Furthermore special provisions will be required on those projects where a threshold value of the decision criteria has been exceeded. Threshold values are defined as values that, if exceeded, render the pavement totally unacceptable and in need of rehabilitation.

BEHAVIOR

Behavior is the immediate reaction or response of a pavement to load, environment, and other inputs. Pavement loads are due to a number of factors including those associated with traffic, temperature cycling, and shrinkage and expansion of subgrade soils as well as the materials that make up the structural section.

Behavior in terms of response to load is normally measured in terms of stress, strain, or deflection and can be predicted with an acceptable degree of accuracy from certain mechanistic models. These behavior measurements can be used to determine the validity of certain mechanistic models as well as an indication of the existing load carrying capability of the pavement. Thus an integral part of the development of a pavement rehabilitation strategy must contain a measure of the existing pavement structure behavior that ideally could be used for the following purposes:

1. Pavement inventory in terms of load carrying capability;
2. Input for determination of rehabilitation needs (existing condition in terms of load carrying capability), and
3. Verification of mechanistic models for pavement design and pavement overlay thickness.

Existing Methods of Measuring Behavior

As shown on Table 2, pavement loads may be due to many factors in addition to traffic loads. Ideally, the response of a pavement to all of these individual loads could be measured and mechanistic models developed to predict behavior and performance. In addition, ideal devices to measure critical coupled loads such as traffic and thermal loads are desirable.

The Group 2 report (8) and a review of the literature suggest that methods for measuring behavior of pavements due to loads can be conveniently grouped into the following categories:

1. Bearing tests,
2. Deflection tests,
3. Impact tests,
4. Vibration tests, and
5. Other methods.

Many of the test methods identified measure the response of the pavement in terms of deflection, radius of curvature, and/or elastic modulus under conditions that simulate traffic loads in terms of both load magnitude and frequency of loading. Little attempt is

Table 1. Summary of existing submodels.

Category	Model	Function of Model	Advantages	Disadvantages	Implementation	Research Needs
Safety	Skid Number	To predict decay in skid resistance with traffic	Permits designer to utilize minimum skid numbers as decision criteria	Does not completely recognize the complex interaction of tires, surface, and water	Numerous States Air Force (3)	Prediction of SN Decay with Nt'l. Prop.
	Accident Data	Tie accident data to safety characteristics		Difficult to ascertain causes of accidents	(1)	Better accident identification
	Maintenance Safety Index	Relate maintenance activities exposure with the probability of an accident	Permit criteria as to the strategy and the degree of exposure by maintenance employees		Concept in California (1)	Develop the concept
Structural	Elastic Layered	Predict stresses in pvt. structure due to load	Provide better criteria for design of the layers	Lack of adequate transformation between stress and aerial distress	Used for guidance in design (2)	Transformation between stress and perf.
	Deflection	Ties deflection of pvt. structure to performance	Easy to implement and in place evaluation of an existing pvt. structure	Material properties can influence the deflection-perf. relationship	Used as design criteria by many agencies (3)	Transformation between deflection and perf.
	Thermal Fracture	Predict stresses developed in pvt. by shrinkage forces due to thermal changes	Provides a way to design against low temp. cracking	Lack of adequate transformation between stress and aerial cracking	Canadian Provinces (2)	Transformation between cracking and perf.
	Reflection Fracture	Predict stresses (cracking) that occurs in overlay due discontinuities in lower layers	Prevent reflection cracking		All State Highway Depts. (2)	Transformation between cracking and perf.
	Structural Number	Predicts performance history of pvt. structure as function of mixed traffic	Provides a transformation between input and aerial distress	Problems in establishing coefficients for materials other than AASHO Road Test	AASHO Interim Guides (3)	Quantify, structural coefficients
Cost	Construction	Compute present worth of all const. and routine maint. costs incurred during life	Permits designer to examine total expenditure during design life		Records available (2)	Procedures for translating present to future
	User	Compiles direct and indirect costs to user resulting from level of service and various activities on pvt. structure	Permits user costs to be considered in decision process	Data required is difficult to obtain and evaluate	Used in SAMP, FPS, and RPS computer programs (2)	Better information
	Noise	Relate noise from surface to a cost item	Permits noise costs to be considered in decision process	Data required is difficult to obtain and evaluate	Only in subjective manner (1)	Relating noise to a composite noise rating
Maintenance	Level of Service	Relates quantity and distribution of traffic to the available resources	Optimum expenditure of funds	Area of political decisions	Subjective only (1)	Develop models
	Performance	Relates the effect of maintenance activities on the pvt. struct. performance	Permits a more reliable comparison of alternate strategies	Many types of activities, therefore difficult to encompass	AASHO Interim Guides and other related procedures (3)	Long term observations
	Drainage	Relate drainage characteristics with pvt. performance	Permits an evaluation of the drainage characteristics of an existing pvt. structure	See above	Indirectly in material characterization (2)	Quantify effect on performance
Traffic	EWL	Converts mixed traffic applications to an objective value	Permits the designer to design for fatigue	Equivalency functions will vary depending on mode of obtaining	Numerous design methods (3)	Better traffic prediction techniques
	Traffic Flow	Computes the capacity of a facility for various methods of handling traffic	Permits the optimization of traffic handling		Basis for developing User Cost	Develop models
Optimization		Combines effects of all other models into one basis of comparison	Permits better basis for decision making		Only in component parts (1)	Practical methods to reduce computation time

Table 2. Non-traffic-associated loads.

Pavement Layer	Material	Possible Loading Mechanism
Surface	Asphalt concrete, portland cement concrete, surface treatment	Thermal, absorption, loss of moisture, loss of volatiles, reflection cracking
Base	Untreated, cement-treated, lime-treated, asphalt-treated	Thermal, absorption, loss of moisture, loss of volatiles, reflection cracking
Subbase	Untreated, cement-treated, lime-treated	Thermal, loss of moisture, reflection cracking, freeze-thaw
Subgrade		Loss of moisture, gain of moisture, consolidation, compaction, freeze-thaw

made to measure behavior due to the non-traffic-associated loads described briefly in Table 2.

Although soil classification and soil strength index methods do not directly measure response to load, they can be used as an index with a structural model to predict load carrying capability of the pavement. Advantages, disadvantages, speed of operation, and information indicating where the various devices have been implemented are contained in Table 3.

<u>Bearing Tests</u> -- Bearing tests are distinguished by the application of a static or dynamic load to a pavement or soil through a plate. The static plate bearing test widely utilized to determine modulus of subgrade reaction and deflection is the most common test of the group. The California bearing ratio (CBR) and bearing tests developed in North Dakota and Florida, although not widely used to measure pavement behavior, have been included. Dynamic plate load tests have been used as a research tool but have not gained widespread implementation.

<u>Deflection</u> -- Benkleman beam, California traveling deflectometer, and Lacroix deflectometer are examples of deflection measuring devices. These devices are also capable of obtaining the radius of curvature of the deflection basin. The widespread use of these devices has spread throughout the world.

<u>Impact</u> -- Impact devices employ a dropping weight to impart a shockwave into the pavement structure, which is measured and converted to deflection. Little if any full scale implementation of this category of devices has taken place.

<u>Vibratory</u> -- Vibratory devices have received widespread use as research tools in the last 10 years and in the case of the Dynaflect and road rater acceptance for measurements used in pavement rehabilitation. These devices use either a heavy or light vibrator to excite the pavement. Deflection, radius of curvature, and elastic moduli can be determined.

Several sophisticated wave propagation devices have been grouped under this heading and are used to evaluate airport pavements. Wide frequencies are used to propagate waves into the pavements under relatively heavy loads. The U.S. Air Force and U.S. Army Corps of Engineers are developing such devices.

<u>Other Methods</u> -- Soil classification methods including those used by the FAA are part of pavement overlay design models.

In situ trandducers, although utilized for research only at this state of development, may become important for certain urban facilities. The development of devices of this nature to measure such items as remaining fatigue life and the behavior of pavements subjected to coupled loads appears reasonable with additional research.

Curvature measuring devices have been used in South Africa, California and Texas.

<u>Adequacy of Existing Equipment and Implementation</u>

As indicated previously, equipment to measure pavement behavior should be capable of measuring the existing condition of the pavement as well as supplying input to the pavement overlay design method. Ideally, equipment to measure the existing pavement behavior should have a high speed of operation and simplicity of operation and be easily transportable and capable of obtaining and summarizing mass data. Equipment currently considered implementable for use in mass inventory is as follows:

1. Benkleman beam,
2. Lacroix deflectometer,
3. California traveling deflectometer,
4. Dynaflect, and
5. Road rater.

Certain advantages and disadvantages of the above-mentioned devices are shown in Table 3.

Devices suitable for supplying input to pavement overlay design methods ideally should have a relatively high speed of operation and be simple to operate easily transportable, and capable of obtaining accurate and meaningful results compatible with the design methods. Devices currently considered implementable are

1. Plate bearing
2. Deflection measuring equipment
3. Dynaflect, and
4. Certain vibratory equipment.

Certainly improvements can be made in all of these devices; however, implementation is possible based on existing knowledge.

A value judgment indicating implementation of devices is given in Table 3. This table indicates that the existing information can be used to measure pavement behavior due to traffic loads. Existing knowledge to measure behavior due to non-traffic-associated loads is not adequate.

Research Needs

Research needs for the various categories of behavior measuring devices are given in Table 3. Research needs should be separated into categories recognizing the specific purpose of the equipment. For example, the development of high speed equipment to survey pavement behavior on a mass inventory basis is necessary, but speed is not so critical as demand for equipment to determine pavement behavior for input to a pavement overlay design procedure. Ideally, equipment that could satisfy both of these needs as well as provide a check for pavement design models should be considered a research need.

A general research need is the development of detailed diagnostic equipment that will measure the following parameters at a high speed of operation and that is easily transportable, capable of accepting, analyzing, storing, and retrieving large amounts of data, and easily operated:

1. Elastic moduli,
2. Failure stresses and/or strains,
3. Fatigue properties of materials,
4. Thickness of pavement layers, and
5. Response of non-traffic-associated loads.

Wave propagation and wave velocity measuring systems including ultrasonic, microwave, remote sensing, laser beams, and photographic techniques appear to be worthy of additional research.

Current Research

Research is being conducted on wave propagation devices by the Air Force and Corps of Engineers. Vibratory equipment has received a great deal of research effort in the last 10 years. Additional research is now in progress.

The development of faster, more reliable deflection measuring devices continues while research on impact devices and bearing tests is limited.

Current research on in situ transducers is largely limited to the aerospace industry with some application in short sections of in-service pavements.

DISTRESS

Roadway distress manifestations result when some limiting response or damage occurs in the roadway. Distress manifestation associated with the pavement is usually in the form of pavement cracking, distortion, and disintegration; vehicle and pavement related noise; and skid resistance. Numerous methods have been established to measure pavement distress, and attempts have been made to establish limiting values of the various forms of distress. Complications associated with establishment of these limiting responses are due in part to the inadequacies of distress measuring methods to identify and accurately measure the significant factors. In addition, distress affects not only the operator and passenger of the vehicle but also the operation of the vehicle and the people and goods adjacent to the roadway.

The measurement of pavement distress can be obtained from user-oriented or mechanistic evaluations. The output from the subjective user evaluations and the objective mechanistic evaluations taken at any particular time is usually referred to as the level of service. The history of this level of service, or serviceability, with time is a measure of pavement performance.

Mechanistic evaluations are concerned with measuring in quantitative terms items such as pavement cracking, road roughness, and skid resistance.

User-oriented evaluations such as the present serviceability rating are often intended to measure only the riding quality provided by the pavement. A measurement at any particular time is the level of service provided to the user. Variations of this present serviceability with time are a measure of pavement performance. The best known definitions and procedures for measuring serviceability in North America are those developed at the AASHO Road Test and in Canadian pavement evaluation studies. The AASHO terminology for perforance rating is present serviceability rating (PSR), whereas the Canadian equivalent is presently referred to as the riding comfort index (RCI) to more explicitly denote the evaluation only of pavement riding quality.

It is obviously impractical in terms of both time and expense to evaluate performance serviceability on anything but a limited basis by using the rating panel method. Consequently, considerable effort has gone into correlating various mechanical evaluation methods with panel performance evaluations. This effort has led to the development of a number of road roughness measuring devices inasmuch as roughness is generally considered to be of major importance to the user and is thus reflected in his performance evaluations.

In addition to road roughness measuring devices mainly used to correlate with user-oriented evaluation, the engineer has been concerned with pavement distress in terms of cracking, distortion, and disintegration. Pavement evaluation methods attempting to measure these forms of distress are referred to as condition surveys. Other methods of pavement evaluation are those associated with measuring pavement bahavior. These test methods along with analysis techniques evaluate the structural load carrying capacity of a pavement among other factors but do not measure mechanistically the pavement structural distress.

Two other forms of pavement related distress are of concern to the engineer and the driving public: (a) highway noise created largely by the tire pavement interaction and the vehicle and (b) skid resistance and other safety related measurements.

Table 4 presents a brief summary of groups of methods used to measure pavement distress. It is the function of pavement evaluation methods to measure the above-mentioned distress in order to (6)

1. Provide data for checking design predictions and updating them as necessary,
2. Reschedule rehabilitation measures as indicated by updated predictions,
3. Provide data for upgrading the design models themselves, and
4. Provide information for updating network rehabilitation progress.

Pavement evaluation thus serves the planning and design activities of pavement management and is, therefore, a part of the pavement management system and provides the means for assessing rehabilitation needs on both a project and a network basis.

Existing Methods of Measuring Pavement Roughness

Pavement roughness evaluation has received considerable attention from most highway and airport agencies inasmuch as roughness is generally accepted as the primary component of serviceability as viewed by the user. A number of attempts have been made to correlate different roughness measuring devices with panel user oriented performance ratings. These results are presented in the literature together with descriptions of the numerous devices used to measure roughness.

Table 5 is based on a literature review and the workshop proceedings (6,10). For convenience, road roughness measuring devices have been grouped into the following categories:

1. Profilometer,
2. Mechanical vibrometer, and
3. Precise leveling.

Speed of operation, advantages, disadvantages, research needs, and agencies that implemented the individual devices have been identified.

Table 3. Behavior.

CATEGORY	METHOD	BEHAVIOR MEASURED	SPEED OF OPERATION (MEASUREMENTS PER DAY)	ADVANTAGES	DISADVANTAGES	IMPLEMENTATION	RESEARCH NEEDS
Bearing	Static Plate Bearing	•Deflection •Elastic Modulus •Modulus of Subgrade Reaction	1 to 10	•History of Use and Acceptance	•Slow Test to Perform •Massive Equipment Necessary	•U.S. Navy •Corps of Engineers •U.S. Army •U.S. Air Force (3)	•Development of More Portable Equipment •Increase Speed of Data Collection
	Dynamic Plate Load	•Deflection •Elastic Modulus	1 to 10	•Dynamic Loads Which Simulate Traffic	•Slow Test to Perform •Massive Equipment Necessary	•Research Tool Only (1)	•Development of More Portable Equipment •Increase Speed of Data Collection
	California Bearing Ratio	•Soil Strength Index •Deflection •Elastic Modulus	1 to 5	•History of Use and Acceptance	•Slow Test to Perform •Not Suitable for Stabilized Materials	•U.S. Army •U.S. Air Force •Corps of Engineers (3)	•Increase Speed of Data Collection
	North Dakota Florida? Other	•Soil Strength Index	1 to 5	•Acceptance Within State	•Does Not Measure Deflection or Elastic Modulus But Soil Strength Index Only •Not Suitable for Stabilized Materials	— (1)	•Increase Speed of Data Collection •Determine Relationship Between Measured Data and Behavior
Deflection	Benkleman Beam	•Deflection •Radius of Curvature	300	•History of Use and Acceptance •Realistic Loads •Simple to Perform •Easily Transportable	•Speed of Operation	•Asphalt Institute •U.S.-Oklahoma and Other States •Great Britain R.R.L. •Some South African Provinces •Canada-RTAC (3)	•Reliability of Measurements •Increase Speed of Data Collection
	California Traveling Deflectometer	•Deflection •Radius of Curvature	1800 to 2000	•Speed of Operation •Realistic Loads •Easily Transportable	•Downgrade Measurements Are Not Possible •Measurements on Curves Not Taken Because of Safety	•California (3)	•Determine Reliable Elastic Moduli from Measurements
	Lacroix	•Deflection •Radius of Curvature	5,000	•Speed of Operation •Realistic Loads •Easily Transportable		•France •Great Britain-RRL •Some Canadian Provinces •South African-NIRR (3)	•Determine Reliable Elastic Moduli from Measurements
Impact	State of Washington	•Deflection	—	•Speed and Simplicity of Operation	•Small Load Applied	(1)	•Determine Relationship Between Measured Data and Behavior
	French	•Deflection	500	•Dynamic Load Which Simulates Traffic •Easily Transportable	•Speed of Operation	•Denmark •France (1)	•Determine Relationship Between Measured Data and Behavior •Increase Speed of Data Collection
	Germany Others	•Deflection	—			(1)	•Determine Relationship between Measured Data and Behavior
Vibratory	Shell	•Deflection •Elastic Moduli •Radius of Curvature	5 to 50	•Variable Force and Frequency	•Validity of Certain Elastic Moduli •Small Load Applied •Frequency Range •Speed of Operation	•Research Only (1)	•Increase Speed of Data Collection •Determine Reliable Elastic Moduli from Measurements
	Great Britain RRL	•Elastic Moduli •Deflection	5 to 50	•Variable Force and Frequency	•Validity of Certain Elastic Moduli •Frequency Range •Speed of Operation	•Research Only (1)	•Determine Reliable Elastic Moduli for Various Pavement Layers
	French-LCPC	•Elastic Moduli •Deflection		•Variable Force and Frequency	•Validity of Certain Elastic Moduli •Speed of Operation	•Research Only (1)	•Extend Available Load Magnitude and Frequency for Any Single Device
	Road Rater	•Deflection •Radius of Curvature •Elastic Modulus	600-800	•Variable Force •Low Maintenance •Speed of Operation	•Frequency Range •Small Force	•Pennsylvania •California (2)	•Determine Reliable Elastic Moduli from Measurements
	Dynaflect	•Deflection •Radius of Curvature •Elastic Modulus	400-600	•Variable Force •Low Maintenance •Speed of Operation	•Frequency Range •Magnitude of Load Applied	•Texas •Utah •California •Virginia •Pennsylvania (3)	•Determine Reliable Elastic Moduli from Measurements
	Cox	•Deflection •Radius of Curvature •Elastic Modulus	1,000	•Speed of Operation		•Under Development for Contra Costa County, California As Well As Other Agencies (2)	•Determine Reliable Elastic Moduli from Measurements
	U.S. Air Force	•Deflection •Radius of Curvature •Elastic Modulus	300	•Variable Force and Frequency •Large Force •Multipurpose •High Frequency	•Size of Equipment •Slow Test to Perform	•Under Development (2)	•Development of More Portable Equipment
	Corps of Engineers	•Deflection •Radius of Curvature •Elastic Modulus	300	•Variable Force and Frequency •Large Force •Multipurpose	•Size of Equipment •Slow Test to Perform	•Under Development (2)	•Development of More Portable Equipment
Other Methods	Soil Classification	Soil Support	1 to 5	•History of Use	•Does Not Measure Behavior of Entire Pavement But Only Subgrade	•FAA •States Using Group Index Method, etc. (3)	•Determine Relationship Between Measured Data and Behavior
	In-Situ Transducers	•Deflection •Deformation •Load		•Capable of Measuring Behavior Under Actual Pavement Loading Conditions •Measure Coupled Loads Due to Traffic and Environment	•Installation •Reliability and Maintenance •Permanent Installation •Limited Number of Measurements Possible	•Research Only (1)	•Develop Method of Measuring Remaining Life of Pavement
	Curvature	•Radius of Curvature	300	•Small •Transportable	•Require a Slow Moving Load	•South Africa •California (3)	•Increase Speed of Measurements

RRL - Road Research Laboratory
LCPC - Laboratoire Central Des Ponts Et Chaussees
RTAC - Road and Transport Association of Canada

Table 4. Pavement evaluation methods.

Evaluation Group	Examples of Categories	Comments
Performance	Present Serviceability Index Riding Comfort Index	•User-oriented subjective evaluation mainly of road roughness
Roughness	Profilometer Mechanical Vibrometer Precise Level	•Developed primarily to provide faster and more economical performance evaluations
Condition	Visual Photographic	•Measurement of type, degree, magnitude and location of pavement distress
Noise	Sound Recording Equipment	•Measurement of noise associated with tire-pavement interaction and vehicle and the effect of the noise on the driver and adjacent people and
Skid Resistance	Indirect - Surface Textile Direct - Locked-wheel Skid Trailer	•Measurement of accident potential of given pavement section
Behavior	Bearing Deflection Impact Vibratory Wave Propagation	•Measurement of the immediate reaction or response of a pavement to load
Cost	Maintenance Cost per mile Benefit - Cost Index USER Cost	•Measure of maintenance affectiveness, benefits of certain maintenance activities, feedback to mechanistic pavement design models, etc.
Traffic	Traffic Volume Capacity	•Measurement of pavements ability to adequately handle present and future traffic

Profilometer -- Rolling straight edge measuring equipment was used in the United States as early as 1900. Since that time, numerous profile measuring devices identified by such names as Viagraphs, profilograph, and profilometers have been developed and utilized by highway and airport agencies.

Correlation of user-oriented performance evaluation with roughness measurements was formalized at the AASHO Road Test. The Chloe profilometer was utilized, in part, for this correlation.

Surface dynamic profilometers have received increased use in the last 10 years as research tools and for calibration of other roughness measuring equipment. Surface dynamics profilometer equipment promises to be the most desirable method of this category of equipment to measure road profile characteristics. Its major advantages are

1. Determination of actual profiles.
2. Capability of handling large amounts of data by automated means,
3. Operating speeds sufficient to cover reasonable amounts of pavement in a reasonable time,
4. Capability of detecting and analyzing longer wave lengths in the pavement,
5. Excellent repeatability, and
6. Capability of use for calibration of car road meters.

Mechanical Vibrometer -- This category of equipment measures vertical movement between the axle of an automobile or a wheel in the case of trailer devices and the mass automobile or wheel supports. The State of New York developed a device called a Via-Log prior to 1926. This device measured the vertical movement between the front axle and the body of the car. Similar devices commonly referred to as the PCA road meter, the Mays ride meter and the Cox and Son road meter have been developed by using many of the same principles. The major advantages offered by this newer equipment are in terms of improved measuring and recording equipment, thus allowing higher speeds of operation.

Limited work has been performed on measuring runway and taxiway roughness with instrumented aircraft. Certainly this is an area that deserves further consideration from both a vehicle operational standpoint and a passenger standpoint.

In 1941 the Bureau of Public Roads reported the development of a trailer unit capable of measuring road roughness. This device, known as the BPR roughometer, has been widely used and correlated with performance evaluations. Excellent repeatability and possible use as a calibrator for other roughness measuring devices make its use attractive.

This category of roughness measuring devices does not give a reliable measure of roughness wave length.

Precise Leveling -- The precise leveling method has been utilized for a number of years. A survey rod and level have been widely used on airfields and some highways. Research on the application of laser beams together with a traveling rod have been reported that will offer a faster and perhaps more reliable method.

Adequacy of Existing Roughness Measuring Equipment and Implementation -- Equipment to measure road roughness would ideally have the following characteristics:

1. Correlation with performance evaluation,
2. Measurement of wide spectrum of roadway wave length,
3. Measurement response of typical vehicle traffic on the facility,
4. High speed of operation,
5. Simple and easy operation,
6. Capability of mass inventory and of measuring, analyzing, storing, and retrieving large amounts of data, and
7. Transportability.

Such equipment has not been developed in a single unit to date. However, the combination of several existing devices will provide the necessary measurement system.

The combination of equipment to meet the desired requirements would consist of a surface dynamics profilometer or precise leveling device for correlation with performance evaluations and for calibration purposes. Car road meters such as the PCA, May, or Cox would be utilized for the correlation of mass inventory data. Correlation of these car meters with performance evaluations would be maintained through profilometer equipment such

Table 5. Pavement mechanistic-roughness evaluation.

CATEGORY	METHOD	QUANTITY MEASURED	SPEED OF OPERATION	ADVANTAGES	DISADVANTAGES	IMPLEMENTATION	RESEARCH NEEDS
Profilometer	Rolling Straight Edge (Calif. U of Michigan, Illinois, French, Others)	Vertical Movement	Slow	•Repeatability	•Operating Speeds •Measurement of Certain Wave Lengths	•California Division of Highways •University of Michigan •Other Agencies (3)	•Increase Speed of Operation and Measurement of Certain Wave Lengths
	CHLOE Profilometer	Slope Variance	Slow	•Repeatability	•Slow Operating Speed •Measurement of Long and Short Wave Lengths •Movement of Towing Vehicle	•AASHO Road Test •General States (3)	•Increase Speed of Operation and Measurement of Certain Wave Lengths
	British-RRL	Vertical Movement (Inches Per Mile)	Slow	•Repeatibility •Calibrations of Other Roughness Measuring Devices	•Slow Operating Speed •Measurement of Long Wave Lengths	•Several Canadian Provinces •Canadian Ministry of Transport •British-R.R.L. (3)	•Speed of Operation and Measurement of Long Wave Lengths
	Surface Dynamics Profilometer	Amplitude and Length of all Waves	Moderate	•Repeatability •Calibration of Other Roughness Measuring Devices •Measurement of Long Wave Length	•High Capital and Operating Costs •Highly Skilled Operating Personnel Required for Operation •Data Reduction Costs •Complexity of System •Not a Direct Measure of Vehicle Ride Characteristics	•General Motors •Texas •Michigan (3)	•Transfer Function for Roadway Wave Length and Frequency to User Opinion
Mechanical Vibrometer	VIA-Log	Relative Vertical Movement Between Rear Axle and Mass (Body of Car)	Traffic Speed	—	—	•Developed in 1926 and Utilized in New York State (2)	
	PCA	Relative Vertical Movement Between Rear Axle and Mass (Body of Car)	Traffic Speed or 50 MPH	•Low Cost •Simplicity and Ease of Operation •Speed of Operation •Mass Inventory Possible •Portability of Equipment	•Repeatability •Affected by Environment •Does Not Measure True Amplitude or Length of Waves	•Wisconsin •Washington •Minnesota •California (3)	
	Mays Ride Meter	Relative Vertical Movement Between Rear Axle and Mass (Body of Car)	Traffic Speed or 50 MPH	•Low Cost •Simplicity and Ease of Operation •Speed of Operation •Mass Inventory Possible •Portability of Equipment •Continuous Record	•Repeatability •Affected by Environment •Does Not Measure True Amplitude or Length of Waves	•Texas (3)	•Improve Repeatability of Results •Identify Significant Vehicle and Environmental Factors Affecting Roughness Measurement •Improve Data Handling Techniques
	Cox and Son	Relative Vertical Movement Between Front Axle and Mass(Body of Car)		•Low Cost •Simplicity and Ease of Operation •Speed of Operation •Mass Inventory Possible •Portability of Equipment •Continuous Record	•Repeatability •Affected by Environment •Does Not Measure True Amplitude or Length of Waves	•Research Activities (2)	
	BPR Roughometer	Relative Vertical Movement Between Wheel and Mass (Trailer)	20 MPH	•History of Use	•Low Operating Speed •Attenuation of Wave Lengths in the Ride Frequency Range •Repeatability and Constancy Related to Calibration	•Several States (3)	
Precise Leveling	Rod and Level	Amplitude and Length of All Waves	Slow	•Precise Measurement •History of Use	•Slow Operating Speeds •Safety •"Down Time" of Facility •Not a Direct Measure of Vehicle Ride Characteristics	•Agencies Associated with Airfields (3)	•Increase Speed of Operation •Transfer Function for Roadway Wave Length and Frequency to User Opinion
	Traveling Rod and Laser Beam	Amplitude and Length of All Waves	Slow	•Precise Measurement	•Slow Operating Speed •"Down Time" of Facility •Not a Direct Measure of Vehicle Ride Characteristics	•Under Development (2)	

Table 6. Areas of applicability for various types of roughness measuring equipment.

Type of Facility	Construction Monitoring	Mass Inventory
Expressway or Primary Highway	BPR Roughometer Car Ride Meters Surface Dynamics Profilometer Rolling Straight Edge (British Road Research Laboratory) (CHLOE Profilometer)	Car Ride Meters Surface Dynamics Profilometer (British Road Research Laboratory) (CHLOE Profilometer)
Secondary (Rural) Highway	BPR Roughometer Car Ride Meter Rolling Straight Edge (Surface Dynamics Profilometer) (British Road Research Laboratory) (CHLOE Profilometer)	Car Ride Meters (Surface Dynamics Profilometer) (British Road Research Laboratory) (CHLOE Profilometer)
Country or Local Rural Highways	BPR Roughometer Car Ride Meters Rolling Straight Edge (Surface Dynamics Profilometer)	Car Ride Meters
Airfields	Car Ride Meters Surface Dynamics Profilometer British Road Research Laboratory (Precise Level)	Car Ride Meters Surface Dynamics Profilometer British Road Research Laboratory (Precise Level)

1. Brackets denote applicability primarily for special purposes or control sections

as the surface dynamics profilometer device being developed in Texas or precise leveling methods as described above. An alternate method to calibrate car load meters is vibration tables or other suitable devices. Repeatable road profiles are programmed into the vibration table. The road meter response of the automobile excited by this table would be measured at periodic intervals.

As noted, most of the devices noted in Table 5 are considered implementable; however, the areas of applicability of various roughness measuring devices are shown in Table 6. In addition it should be noted that roughness measurements can be utilized for construction monitoring, maintenance programming, inventory and network programming, and research.

Current Research

Current research on pavement roughness in the highway field is aimed at establishing adequate correlation between performance evaluation and rapid roughness measuring equipment such as the car ride meter and the surface dynamics profilometer. Additionally, research is being conducted to establish the sensitivity of certain vehicle characteristics and environmental characteristics that affect car ride meter measurements. Methods of calibrating car ride meters are also being considered.

Research Needs -- Development of a single item of equipment to meet the ideal characteristic for roughness measuring equipment described above should be pursued. The most important items are speed of operation, simplicity of use, and ease of data handling for mass data collection.

What excessive highway and airfield roughness is and the characteristics of this roughness should be determined. Certainly other specific research needs could be identified, but in general these items appear to be of the greatest importance.

Pavement Condition Surveys

Most highways and airport agencies conduct periodic pavement condition surveys on selected sections, or on a mass basis. These surveys are measurements of pavement distress such as cracking, distortion, and disintegration and can be defined as any process of identifying either qualitatively or quantitatively visible manifestations of pavement distress.

Condition surveys are conducted in a variety of fashions and to varying degrees of accuracy, subjectivity, and reliability by the many agencies employing such surveys. In general, condition surveys are conducted for the following purposes:

1. To be used as input to development of a structural rating or index,
2. To aid in projection of budget requirements,
3. To aid in decisions to perform or not perform maintenance,
4. To act as a diagnostic tool for assessment of design and/or construction procedures,
5. To be used as input to rehabilitation design, and
6. To be used as input in determining pavement performance history (20).

As indicated, these purposes closely coincide with those given above for the establishment of evaluation methods for pavement distress.

Existing pavement condition survey methods have been classified into two broad categories identified as visual and photographic on Table 7. Details of the items measured in condition surveys conducted in Washington, Minnesota, and Ohio and by the Canadian Department of Transport, British Research Laboratory, and King County, Washington, are summarized together with advantages and disadvantages of each method. All of the visual methods obtain a subjective measure of the type, degree, and magnitude of distress in addition to its approximate location.

Nomenclature used by most agencies in describing pavement distress types is somewhat uniform although the refinement varies among procedures. The following major categories of distress are usually recognized by the various mathods:

1. Cracking (alligator, longitudinal, transverse, map, reflection),
2. Disintegration (raveling, stripping, spalling, scaling),
3. Permanent deformation (rutting, faulting), and
4. Distortion (settlement, heave).

Table 7. Pavement mechanistic evaluation-condition survey (distress measurements).

CATEGORY	METHOD	MEASUREMENTS	ITEMS MEASURED	ADVANTAGES	DISADVANTAGES	IMPLEMENTATION	RESEARCH NEEDS
Visual Evaluation	Washington State (Flexible)	Type, Degree, Magnitude and Location of Distress	°Rutting °Waves, Sags, Humps °Corrugations, Potholes, Raveling, Flushing °Alligator Cracking °Longitudinal Cracking °Transverse Cracking °Patching	°History of Use and Acceptance	°Subjectivity of Measurements °Speed °Safety	°Washington °California (3)	°Improve Repeatability of Results °Increase Speed of Data Collection
	Washington State (Rigid)	Type, Degree, Magnitude and Location of Distress	°Cracking °Raveling, Disintegration, Pop Out Scaling °Joint Spalling °Pumping, Blowing °Blowups °Faulting, Curling, Warping, Settlement °Patching	°History of Use and Acceptance	°Subjectivity of Measurements °Speed °Safety	°Washington °California (3)	°Improve Repeatability of Results °Increase Speed of Data Collection
	Minnesota (Flexible)	Type, Degrees, Magnitude and Location of Distress	°Rutting °Alligator Cracking °Longitudinal Cracking °Transverse Cracking °Multiple Cracking °Patching	°History of Use and Acceptance	°Subjectivity of Measurements °Speed °Safety	°Minnesota (3)	°Improve Repeatability of Results °Increase Speed of Data Collection
	Minnesota (Rigid)	Type, Degree, Magnitude and Location of Distress	°Spalled Joints °Faulted Joints °Cracked Panels °Broken Panels °Faulted Panels °Patching	°History of Use and Acceptance	°Subjectivity of Measurements °Speed °Safety	°Minnesota (3)	°Improve Repeatability of Results °Increase Speed of Data Collection
	Minnesota (Bituminous Overland)	Type, Degree, Magnitude, and Location of Distress	°Longitudinal Cracking °Transverse Cracking °Multiple Cracking °Patching	°History of Use and Acceptance	°Subjectivity of Measurements °Speed °Safety	°Minnesota (3)	°Improve Repeatability of Results °Increase Speed of Data Collection
	Ohio (Proposed)	Type, Degree, Magnitude, and Location of Distress	°Deterioration °Obstruction °Flushing °Stripping	°History of Use and Acceptance	°Subjectivity of Measurements °Speed °Safety °Few Items Measured °Limited Measures of Degree and Magnitude of Distress	°Ohio (Research) (2)	°Improve Repeatability of Results °Increase Speed of Data Collection
	Canadian Dept. of Transport (Flexible)	Type, Degree, Magnitude, and Location of Distress	°Hair Cracking °Alligator Cracking °Longitudinal Cracking °Transverse Cracking °Chicken Wire °Map Cracking °Reflection Cracking °Stripping °Raveling °Rutting °Deformation °Distortion °Subgrade Settlement °Skin Patches °Deep Patches °Localized Reconstruction °Frost Heave	°History of Use and Acceptance	°Subjectivity of Measurements °Speed °Safety	°Canadian Dept. of Transport (3)	°Improve Repeatability of Results °Increase Speed of Data Collection
	British Road Research Laboratory	Type, Degree, Magnitude, and Location of Distress	°Disintegration, Flushing °Deformation °Texture °General Variability °Overall Condition	°History of Use and Acceptance	°Subjectivity of Measurements °Speed °Safety °Few Items Measured	°British Road Research Laboratory (3)	°Improve Repeatability of Results °Increase Speed of Data Collection
	King County, Washington (Proposed)	Type, Degree, Magnitude, and Location of Distress	°Corrugation, Shoving, Slippage °Flushing °Raveling °Rutting °Alligator Cracking °Longitudinal Cracking °Transverse Cracking °Waves, Sags, Humps °Patching	°History of Use and Acceptance	°Subjectivity of Measurements °Speed °Safety °Few Items Measured	°King County (Research) (2)	°Improve Repeatability of Results °Increase Speed of Data Collection
Photographic	British Columbia	°Photographs °Pavement Cross Slope °Road Roughness	°Cracking °Some Distortion °Safety Hazards	°History of Use and Acceptance °Speed of Operation °Safety °Combine Several Measurements in Single Operation °Continuous Record	°Subjectivity of Measurements °Speed °Safety °Detail of Pavement Defects	°British Columbia (3)	°Improve Repeatability of Results °Increase Speed of Data Collection
	Washington State	°Photographs		°History of Use and Acceptance	°Subjectivity of Measurements °Speed °Safety	°Washington (Research) (2)	°Improve Repeatability of Results °Increase Speed of Data Collection

Details of individual methods are given in Table 7 to give the reader a better understanding of the differences in condition surveys. In addition to the detail associated with types of distress, some agencies require different forms for flexible, rigid, and overlaid pavements, whereas others use a single form.

Length of sections surveyed and weighting factors assigned various types of distress by the different agencies also vary among those conducting condition surveys.

Photographic methods for conducting condition surveys have been developed in British Columbia and Washington. The British Columbia device gives not only an indication of pavement cracking and distortion but also a road roughness measurement, pavement cross slope, the presence of roadsize hazards and signing needs. Detailed pavement distress cannot be recognized from the photographic techniques now used.

Adequacy of Existing Condition Survey Techniques and Implementation -- Review of Table 7 and literature indicates that a number of condition survey methods have been implemented by various agencies; however, certain problem areas have been identified and are given below.

1. Undesirable subjectivity in surveys due to present techniques and human factors,
2. Absence of valid, workable statistical sampling procedures for highway surveys,
3. Adequate delineation of established survey areas for repetitive survey purposes,
4. Lack of uniformity in severity weighting techniques for distress types,
5. Inability with current data storage and retrieval methods to achieve a valid, workable inventory of pavement condition, and
6. As currently conducted, hazardous to survey personnel and disruptive to traffic on urban freeways.

In spite of these problem areas, the state of the art is such that implementation should proceed.

Current Research -- Current research efforts dealing with the development of condition surveys for particular agencies are continuing. Emphasis is being placed on not only the condition of the roadway, but also the shoulder and roadside from both safety and aesthetic standpoints.

Data analysis, storage, and retrieval systems are also being investigated for handling condition survey data as part of pavement management systems.

Research Needs -- The following research needs have been developed based on a review of the literature and the workshop proceedings:

1. Develop uniformity in condition survey techniques and procedures by identifying common survey procedures that may be standardized on a national or at least a regional basis and by devising an implementation program to include procedural manuals and possibly training centers similar to the FHWA regional skid calibration centers.
2. Develop mechanistic objective techniques and equipment for condition surveys. Suggested areas of investigation might include holography (either visual or acoustical), modified radar technique, infrared sensing techniques, and photographic techniques. An ideal objective for this research area would be the development of a universal testing vehicle capable of conducting pavement skid, roughness, and condition surveys sumultaneously and at high speeds.

Another research need is definition of the relationship between distress and performance. There is a need to develop both pavement condition or distress indexes and serviceability/performance relationships. This work should develop methods for establishing proper weighting functions for combining the various types of distress (roughness, condition, skid resistance, noise) into a common index and a method or relating this distress index to the user-oriented present serviceability index or other suitable measure of performance.

Also condition surveys need to be developed that not only measure pavement distress, but also consider the condition of shoulder, roadside, drainage, and traffic related items for use in maintenance management.

Measurement of Skid Resistance

The measurement of the roadway for safety usually considers only slipperiness (in terms of skid resistance); however, several components of safety should be included to make a complete analysis: those affecting the roadway, the driver, and the vehicle. A review of research indicating the significance of the driver and vehicle as well as a complete description of the interactions among the driver, the pavement, and the vehicle is not the purpose or intent of this report. The factors affecting roadway safety are however important to pavement rehabilitation, and ideally one should consider measurements of the following roadway characteristics:

1. Skid resistance,
2. Ruts (as they relate to accumulation of water and the dangers of hydroplaning or ice accumulation),
3. Visibility as affected by light reflectivity and the pavement surface.
4. Pavement surface texture and its effect on skid resistance and the potential to hydroplane,
5. Lane demarcation,
6. Debris and foreign objects, and
7. Roadside hazards.

The current state of the art is such that reliable effective measuring systems have been developed to measure skid resistance, and to a certain degree hydroplaning potential of a particular pavement section can be determined by careful measurements of rut depths, pavement surface texture, cross slope, rainfall intensity, and coefficient of friction at various speeds. Adequacy of methods to appropriately measure light reflectivity and lane demarcation and relate these measurements to highway safety is lacking. Some effort has been expended on the recognition and establishing the severity of roadside hazards such as cut and fill slope, drainage structures and other structures, and signing.

Because skid resistance measurement systems appear to be implementable at this time and because they are considered to be of vital importance for determining overlay strategy, a brief review of these methods is presented.

Evaluation of the skid resistance of a pavement section can be measured by indirect or direct methods, which result in determination of a coefficient of friction. Table 8 presents a summary of existing methods for measuring skid resistance.

Indirect methods include analysis of accident data to determine location and frequency of accidents and accident costs in terms of personal injury and vehicle damage. Review of accident data offers the opportunity to obtain data on in-service facilities where a critical combination of pavement, vehicle, and driver inadequacy resulted in the accident rather than a single element such as coefficient of friction. Other indirect methods for determining skid resistance are by surface texture measurements using techniques such as the sand patch or silly putty and photography.

Direct methods of measuring skid resistance include the British portable tester, vehicle devices, locked-wheel trailers, "slip" testers, and "yaw" testers. All of these methods determine a coefficient of friction in a certain manner. Thus when skid numbers (coefficient of friction) are reported and associated limiting values of distress are used, the type of testing devices, speed of operation, and water depth should be reported.

Advantages and disadvantages of the various instruments are given in Table 6 and the application of the British portable tester to areas that are not easily acceptable to the other types of testers should be recognized.

Adequacy of Existing Skid Resistance Measuring Techniques and Implementation -- Review of Table 8 indicates that many of the devices are in use on a widespread inventory and reserach basis. In particular, the ASTM # 274 locked-wheel trailer is receiving widespread use for inventory purposes in many states, and thus one can conclude that it adequately performs its intended purpose of obtaining meaningful data, at a high speed of operation, and is capable of obtaining, analyzing, and storing mass data in a relatively simple fashion.

Methods to predict hydroplaning, however, are difficult to implement at this point because of the amount and detail of data required.

Current Research -- Current research in the area of skid resistance deals with the development of devices to measure friction values and determine limiting values of friction that are necessary to maintain vehicle control under a variety of vehicle movements. Calibration of skid resistance measuring equipment is receiving widespread attention together with the development of automated skid measuring equipment to reduce the data collection, analysis, and retrieval time. The identification of roadside hazards and survey methods to collect these data is being developed.

Research Needs -- Evaluation methods to identify locations where the potential for hydroplaning is high should be improved. This research effort may require additional work to determine at what pavement length hydroplaning will occur at a given water film thickness and vehicle speed.

Pavement management systems should be made compatible such that existing accident data can be utilized. These data together with items such as surface texture and skid resistance would be valuable for determining rehabilitation strategies.

Measurement of Noise

Highway noise effects have been studied extensively by a number of agencies. These efforts have, however, almost invariably concentrated on outside effects related to the non-user of the pavement. Within-vehicle noise on the other hand is also an important item that must be considered in pavement rehabilitation strategy and should receive increasing attention in the future (6).

Pavement factors that affect vehicle and/or tire-pavement interaction noise include

1. Aggregate surface texture,
2. Pavement surface texture including pavement grooving, and
3. Road roughness.

Certainly, the effect of these variables on noise level is a function of the vehicle and tire characteristics among other factors.

Systems for measuring both within-vehicle noise and noise outside vehicles usually consist of a noise pick-up device and an analyzer and display system. Noise frequency and spectral density measurements can be obtained.

Noise pick-up devices and levels of noise within and without vehicles together with the establishment of tolerable levels appear to be worthy of additional research.

PERFORMANCE

Performance is a measure of the accumulated service provided by a facility and is a measure of the adequacy with which a pavement fulfills its purpose to transport people and goods in a safe and economical manner. Thus performance implies a time-related accumulation of data and is accomplished by conducting user-oriented performance evaluations or mechanistic evaluations on a periodic basis (6).

Based on information presented previously, methods exist for measuring the following data to establish pavement serviceability or its ability to serve traffic in its existing condition:

1. User-oriented performance evaluations,
2. Roughness (distress),
3. Load carrying ability (behavior),
4. Condition (distress),
5. Skid resistance (distress),
6. Noise (distress), and
7. Cost (user and maintenance costs).

The adequacy of existing information to define pavement performance relative to the above categories is summarized in Table 9.

Performance Measurements

User-Oriented Performance Evaluations and Roughness Measurements -- The combination of these two items under one heading is justified because of the widespread use of roughness measurements as indicators of user-oriented performance evaluations.

Since the formal initiation of the concept of pavement serviceability index (PSI) at the AASHO Road Test, various agencies have made routine PSI or roughness measurements on roadways. Unfortunately data have either not been collected for an extended period of time or not reported. PSI as a function of traffic can be obtained from AASHO Road Test information. However, agencies responsible for the operation of highways and airfields do not have sufficient data to make accurate projections on a national or even regional basis in regard to pavement performance from a roughness or user oriented viewpoint. The establishment of limiting tolerable roughness values is also not well documented.

Another major item that requires additional research effort is the effect of minor and major maintenance activities as well as overlaying pavements on pavement performance from a roughness standpoint.

Load Carrying Ability -- The performance of a pavement with respect to its ability to carry load has been discussed. Models relating pavement strength coefficients and pavement structural performance over a period of time have been developed by various states and at the AASHO Road Test. Deflection measuring equipment such as the Benkelman beam have also been used to predict pavement behavior over a period of time. The establisnment of the benefits of minor and major maintenance together with overlay policy has not been extensively investigated. Certain improvements in these methods as discussed are worthwhile research needs.

Conditions Surveys -- The accumulated service of a pavement as determined by a condition survey forms a valuable part of the rehabilitation strategy. Several state highway departments have been collecting detailed information for nearly 8 years. Airfield condition surveys conducted by or for the U.S. Air Force, Navy, and Army have been performed for extended periods of time; however, only limited data are normally collected in these surveys and their usefulness is therefore limited.

The correlation of condition survey data with user-oriented performance evaluations is desirable. It is generally accepted that pavement distress as measured by these condition surveys reaches a severe limit before the pavement reaches a limiting roughness limit in the eyes of the user. Thus a valuable tool may be available if this correlation is established.

The development of maintenance strategy based on the severity of individual or coupled distress manifestations in pavements should be pursued. For example, the existence of several alligator cracks in the wheelpath may call for a heavy overlay, while hairline transverse cracks may suggest a chip seal.

Skid Resistance -- Measurement of skid resistance data on highway and airfield sections has existed at least on a limited scale for several years. These measurements have allowed for the prediction of the decrease in skid number as a function of traffic volume and time from aggregate mineral characteristic and certain laboratory tests such as accelerated polishing tests. While these correlations are not perfect, the engineer does perhaps have more tools to work with than other categories of performance as discussed herein.

Research needs in terms of defining the effect of traffic and surface contaminants on particular pavements should be continued in order that an accelerated laboratory test can be used to predict probable performance with regard to skid resistance.

Hydroplaning involves an important research effort. Methods suitable for mass inventory need to be developed and implemented on a trial basis.

Noise -- The use of noise measurements for the development of rehabilitation strategies has not been practiced on a formal basis. Thus methods to incorporate these measurements and the development of equipment to measure both "within" and "without" vehicle noise should be investigated. Additionally the importance of noise on driver performance

Table 8. Pavement mechanistic evaluation (skid numbers).

CATEGORY	METHOD	ITEM MEASURED	ADVANTAGES	DISADVANTAGES	IMPLEMENTATION	RESEARCH NEEDS
Indirect	Accident Data	°Location of Accident °Cost of Accident	°May Indicate Location Where Vehicle, Driver and Pavement Combine in a Serious Combination	°Data Recording °Data Storage °Data Access	°Several States Including Texas, N.Y., Virginia, Kentucky, Tenn., California (2)	°Data Recording, Storage and Access
	Surface Texture	°Depth of Texture	°Indicates Potential to Hydroplane	°Speed of Operation °Safety °Correlation With Skid Number	°Several Methods Are Implementable °Research in Texas, Kentucky, Tenn., Virginia, Penn., N.Y., Florida (2)	°Increase Speed of Operation °Improve Correlation With Skid Resistance
	Photographic	°Depth of Texture	°Indicates Potential to Hydroplane °Correlation in Specific Areas With Skid Number	°Speed of Operation °Safety °Correlation With Skid Number in General	°Some Methods Are Implementable °Research in Texas and Some Canadian Provinces (1)	°Increase Speed of Operation °Improve Correlation With Skid Resistance
Direct	British Portable Tester	°Skid Number	°Ease of Transport °Can Measure in Localized Area °Laboratory Use	°Speed of Operation °Safety °Correlation With Skid Number	°Research in Virginia, Texas, Florida, Tenn., Kentucky, Penn. (3)	°Improve Correlation With Skid Number at Various Speeds
	Automobile	°Skid Number	°Ease of Transport °Measures Friction Value of Interaction Between the Vehicle and Pavement Desired °Measures Friction at Various Speeds	°Safety °High Costs of Tests When Aircraft Utilized °Variability Due to Vehicle Effects	°Testing With Automobiles Has Been Implemented on Airfields (2)	°Develop Adequate Watering System °Develop Necessary Safety Features
	Aircraft	°Skid Number	°Ease of Transport °Measures Friction Value of Interaction Between the Vehicle and Pavement Desired °Measures Friction at Various Speeds	°Safety °High Costs of Tests When Aircraft Utilized °Variability Due to Vehicle Effects	°Testing With Automobiles Has Been Implemented on Airfields °Aircraft for Research Only (1)	°Develop Adequate Watering System °Develop Necessary Safety Features
	Locked Wheel Trailers (ASTM E 274)	°Skid Number	°Speed of Operation °Measures Friction at Various Speeds °Continuous Record of Data °Cost	°Requires Special Towing Equipment	°23 States (3)	°Develop Adequate Watering System
	"SLIP" Testers (Swedish, FAA, Switzerland, James Brake Decelerometer	°Skid Number	°Measures Maximum Friction Available	°Cost of Equipment °Maintenance	°Canadian Airports (3)	°Develop Adequate Watering System
	"YAW" Testers (Mu-meter, Scrim, U. of Michigan)		°Speed of Operation °Measures Friction at Various Speeds °Ease of Transport °Continuous Record of Data	°High Initial Cost	°Several States (3)	°Develop Adequate Watering System

should be studied.

Costs -- The history of user and maintenance costs is an important element in the development of pavement rehabilitation strategies. User costs associated with vehicle operating costs, travel time costs, accident costs, and discomfort costs have been considered in part in some pavement design models such as SAMP5 and the proposed Texas method. Certainly improvements can be made in these programs. Specifically vehicle operating costs, travel time costs, and discomfort costs due to an increase in pavement roughness can be more accurately summarized. Accident costs have not been considered.

Maintenance cost information has been collected in various forms for extended periods of time. The accuracy of these data is, however, subject to question. Additional detailed information for specific pavement sections should be collected.

COSTS AND SPECIAL PROBLEMS

Costs and the special problems associated with pavement rehabilitation (Table 10) may at times dictate a possible solution or may narrow the solution to a few alternatives. Thus the importance of adequately and accurately considering these factors must be emphasized (1, 6, 16, 17, 18, 19).

Costs

Initial and Rehabilitation Costs -- Initial costs of the facility in terms of protecting the initial cost of the investment should be considered in analysis. Additionally the cost of the selected rehabilitation strategy (rehabilitation cost) should be considered and under normal circumstances should receive considerably more weight than any other single factor in terms of decision criteria.

Maintenance Cost -- We may consider a particular pavement in need of rehabilitation when the cost of maintenance required to keep a pavement above the level of acceptable serviceability becomes excessively high. In field situation when it becomes impossible to maintain a roadway by programmed maintenance funds determined according to predetermined acceptable formulas, a detailed evaluation should be performed.

User Costs -- User costs associated with pavement surface characteristics can be divided into the following components:

1. Vehicle operating costs,
2. Travel time costs,
3. Accident costs, and
4. Discomfort costs.

These costs can be associated with the facility during the rehabilitation period as well as during the life of the facility. For example, as pavement roughness increases, vehicle speeds may decrease thus contributing to travel costs. In addition, road roughness will contribute to discomfort costs, vehicle operating costs, and perhaps accident costs.

The relationship between skid resistance and accidents (and accident costs) is not well established. Additional research is necessary.

Special Problems

Reflection Cracking -- Problems associated with cracks in existing pavements reflecting through new surfacings are important considerations in pavement rehabilitation and can be a key factor in the degree of success obtained in the rehabilitation process. Many techniques and materials have been tried to reduce or eliminate reflection cracking in bituminous overlays. Some of these methods have worked well on particular projects, but none has completely eliminated the cracking and none has been consistently successful in repeated tests (16, 21).

Methods have been developed to effectively control reflection cracking in PCC overlays. One of these methods makes use of a granular base between the old and new pavement.

Research is being conducted at Ohio State University and Texas A&M University to determine the stress and failure modes causing reflection cracking. In addition, FHWA is

Table 9. Pavement performance.

Category	Adequacy of Existing Information	Research Needs
User-oriented performance evaluation	Fair	°Establish performance trends for various types of roads in a number of environmental regions
Roughness	Fair	°Establish benefits of minor and major maintenance operations as well as overlays °Establish limiting criteria
Load-carrying ability	Good	°Establish benefits of maintenance and overlays °Establish seasonal variation of deflection, etc.
Condition surveys	Fair	°Establish correlation between user oriented performance evaluation and condition surveys °Development of maintenance strategy based on type and amount of severity
Skid Resistance and Accident Reports	Good	°Collect data on a wide range of pavement types °Develop methods for predicting hydroplanning suitable for mass inventory
Noise	Poor	°Establish limiting criteria and methods of measurement °Determine significance of noise on driver performance
Cost	Fair	°Additional information on specific types of pavement °Improved methods to consider user costs

Table 10. Costs and special problems.

CATECORY	TYPE	ADEQUACY OF EXISTINC INFORMATION	COMMENTS	RESEARCH NEEDS
Cost	Initial	Good	°Initial Cost of Investment Only	°Determinc Actusl Cost of Specific Items From Unbalanced Bids
	Rehabilitstion Costs	Fair	°This Cost Includes Both Initial Costs of Design and Construction As Well As User Costs	°Develop Better Methods of Simulating User Costs for Civen Situations
	Maintenance Costs	Fair	°Often Includes Costs Associated With Pavement As Well As Other Items	°Develop Methods of Obtaining Specific Maintenance Costs of Interest
	User Costs	Fair	°Costs Include Those Due to Vehicle Operating Costs, Travel Time Costs, Accident Costs, Discomfort Costs	°Develop Methods of Measuring These Costs and Their Relationships With Roughness, Pavement Condition, Skid Resistance, Etc.
	Accident Costs	Good	°Vehicla Damage and Personal Injury Costs	°Reliable Methods of Determining Parsonal Injury Costs
Special Problems	Reflective Cracking	Poor	°No Effective System to Prevent Reflection Cracking in Bituminous Overlays Has Been Reported	°Develop Mechanistic Approach °Develop Corrective Measures
	Thermal Cracking	Fair	°Cracking Due to Fracture of One Thermal Cycle Can Be Predicted and Mixes Designed to Resist This Fracture	°Thermal-fatigue Should Be Investigated
	Drainage	Cood	°Existing Information Allows the Engineer To Design Effective Drainage Systems; However, Costs Are Usually Prohibitive	°Develop Methods That Are More Economical
	Urban Area	Fair	°Problems Associated With Traffic and Time Limitations, Space and Work Limitations, and Environment and Pollution Problems Are Important in Urban Areas	°Develop Effective Coordination Procedures Among the Various Agencies Involved °Develop Pollution Free Methods of Pavement Rehabilitation °Develop Materials and Equipment for Urban Area Rehabilitation
	Airfield	Good	°Difference Between Airfields and Highway Pavements Is Mainly Due to the Magnituda and Number of Loads As Well As the Vehicle Operating Characteristics	°Effect of Pavement Roughness on Aircraft Performance

sponsoring several field trial sections in various states. These trials consist of the use of several types of existing overlay systems purported to reduce reflective cracking.

Research needs associated with reflective cracking include

1. A mechanistic approach to the problem,
2. Criteria and guidelines for evaluating field tests,
3. Improvement of the strength characteristics of bituminous concrete mixtures.
4. Determination of mechanisms involved in reflection cracking, and
5. Development of corrective measures.

Thermal Cracking

Thermal fracture mechanisms of asphalt concrete surfaces are due to the shrinkage forces generated by low temperature which exceed the tensile strength of the pavement material. The thermal cracking problem has been recognized in Canada, and a significant amount of research has led to methods for prediction and materials selection that either reduce or eliminate this form of cracking (17, 22, 23).

Longitudinal and transverse cracking patterns similar to those in Canada exist in the southwestern United States. Maximum low temperatures are above those of Canada and thus suggest that thermal cracking as experienced by the Canadians may not be a problem. Some evidence exists that suggests that this type of cracking may be due to temperature cycling. Further research in the area of thermal fatigue is suggested.

Drainage

Drainage problems to be considered in pavement rehabilitation either occur over large stretches of the project or are localized in short lengths of these pavements. Some of the general drainage problems encountered under and adjacent to pavements include (18)

1. Shallow side ditches,
2. Widening blocking subsurface drainage,
3. Permeable shoulders,
4. Pumping rigid pavements,
5. Impermeability of existing aggregate drainage layers,
6. Reduction of drainage capacity of curbed pavements due to overlays,
7. Raising of drop inlets for overlays, and
8. Drainage of open-graded plant mix seals.

The handling of these problems must be considered in the rehabilitation strategy. A balanced design concept in planning pavement rehabilitation must be considered. For example, a thick overlay may not be necessary for subsurface drainages improved below and adjacent to the pavement. Cost and the potential for success must be compared in determining what strategy to utilize.

Research efforts associated with pavement should be aimed at developing more economical methods and techniques for providing adequate pavement drainages.

Urban Area Problems

Urban area problems associated with pavement rehabilitation can be conveniently placed into three categories: those problems dealing with time and traffic relationships, those caused by space and work relationships, and those caused by the environment and pollution relationships. Typical examples of these problems are given below (19).

Time and Traffic Problems -- High traffic volumes in urban areas restrict maintenance and rehabilitation activities (as well as evaluation surveys) to off-peak hours, severely limiting the working time available to the contractor or maintenance agency. Careful planning of the work schedule and public information accouncements concerning rehabilitation activities are necessary.

In addition to traffic volumes, the weights of the vehicles in urban areas are often in excess of those allowed and should therefore be appropriately taken into account during the overlay design process.

Space and Work Limitations -- The multiplicity of structures and appurtenances found on urban arterial or freeway sections create critical vertical control problems. Overhead clearance, drainage facilities, guardrails, delineators, and increases of dead load on bridges are just some of the items that must be considered.

The combination of the high traffic volume and limited space requirements contribute to detour problems, which makes it difficult to close more than a lane of traffic at any given time. Scheduling a variety of maintenance activities during a single work period is a necessity.

Environment and Pollution Problems -- Noise and air pollution restraints have limited the use of pile drivers, air hammers, and other major construction equipment units. Burning restrictions based on air pollution concerns have prevented the use of heater planers and other techniques for pavement cutting and pavement repairs as well as the use of certain materials in some urban areas.

Of the major special problems discussed, those associated with the environment and pollution appear to have the greatest research need. Additionally the need for the development of materials that allow rapid rehabilitation with little thickness is recognized as well as the development of operations that provide better protection for construction and maintenance crews.

Airfields -- The major differences between airfield and highway pavements are the magnitude and number of loads and vehicle operating characteristics (10).

Because of the weights and operating characteristics of aircraft, significantly larger stresses, strains, and deflections occur at much greater depths. Thus the importance of subgrade drainage and proper joint designs in concrete pavements cannot be overemphasized. Jet blast and turbulence effects together with fuel spillage present additional special problems for airfields.

Skid resistance and roughness are particularly important for certain types of airfields. The potential for hydroplaning is high for certain aircraft because of their high landing speed and high tire pressure. Thus the design of special skid resistance mixes is important.

Pavement roughness can cause discomfort to the pilot and passengers. Excessive roughness can create severe aircraft structural loads as well as difficulty in correctly reading aircraft instruments or gauges.

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A GENERAL FRAMEWORK FOR PAVEMENT REHABILITATION

W. R. Hudson and F. N. Finn

To deal with rehabilitation strategies requires that both the needs of the total highway network for a particular jurisdiction and the needs of a particular project within a given network be considered. For example, standards of performance or allocations of funds will be determined on a network basis, and these decision criteria will eventually influence the availability of funds for rehabilitation of any particular project. Therefore, rehabilitation strategies must include procedures for monitoring the total system at a level adequate to determine the order of priority for major engineering emphasis. Ultimately, evaluation and rehabilitation decisions must be made on a project by project basis. The framework in this report attempts to deal with both network and project requirements, recognizing the need for rational rehabilitation decisions.

To establish criteria for rehabilitation, it is first necessary to establish the need for it. The why of rehabilitation will play a major role in the decision on how the rehabilitation is to be performed. Table 1 enumerates some of the reasons why rehabilitation may be necessary and examples of what might be done in response to the specific condition of the pavement and local decision criteria. Note that one response is to do nothing. This is a viable decision depending on the availability of funds and performance standards for a particular facility.

Table 1 deals primarily with a class of activities generally described as corrective rehabilitation or major maintenance. There are at least two other types of rehabilitation that should be considered: (a) preventive and (b) change in function or strengthening. Proper rehabilitation strategy depends on the engineer's ability to predict a particular need for corrective action or strengthening at a particular time. Preventive rehabilitation can encompass all of the factors included in Table 1.

Under a pavement management framework, it will be useful to consider corrective and preventive rehabilitation under a single term of programmed rehabilitation inasmuch as the pavement management system not only specifies initial construction decisions but also projects future minor maintenance costs and programs as well as rehabilitation overlays. The various criteria for the rehabilitation strategy will include provisions for both corrective and preventive maintenance.

Rehabilitation criteria will be influenced by any current or future plans to change the function of the facility. For example, a pavement that is to be abandoned in a short time would not be eligible for the same level of rehabilitation as one that is to be increased from a secondary to a primary classification.

PAVEMENT EVALUATION FOR PROGRAMMING REHABILITATION OF A HIGHWAY NETWORK

In practice, there is a need for pavement network management and decision models as well as for rehabilitation criteria applicable on a project by project basis. Rehabilitation should be scheduled to correct or prevent the occurrence of an undesirable condition in the pavement.

The logistics for programming rehabilitation requires continuous and systematic monitoring of pavements (sometimes called feedback data) in order to obtain information on the condition, serviceability, and behavior of the pavement (23).

To identify sections in the pavement network that may require rehabilitation requires that preliminary decision criteria for judging the information obtained from the pavement monitoring program be established. For example, a limiting value for the coefficient of friction or for the serviceability level can be established. Violating this limit indicates that maintenance or rehabilitation may be necessary. A procedure for combining the pavement's condition, behavior, and serviceability could also be developed which would be useful in grouping pavements according to probable priorities for rehabilitation. The state of Washington uses this combined index to compare its statewide pavement network. A low rating on the combined scale indicates that a more detailed evaluation is needed.

Special provision should be made for those projects that have violated some threshold value below which their properties are totally unacceptable. A good example of this would be a low skid number. When this condition is reported in the network monitoring program and confirmed by further studies of the project, a recommendation for rehabilitation can be made.

It is pertinent to note that, when rehabilitation is considered mandatory because of one factor, the final recommendation should be made only after a more complete evaluation to determine whether other types of rehabilitation may also be needed.

Figure 1 describes the general concepts relating a network monitoring program to the subsequent detailed project investigation and analysis.

The initial effort requires that the entire network be monitored on a periodic basis. Precisely what evaluation methods and categories are to be included will be a decision for each agency and will depend on the availability of funds, time, personnel, and equipment. It is con-

Table 1. Alternative decisions for rehabilitation or major maintenance.

Reason for Rehabilitation	Rejuvenating or Penetrating Seal	Crack Filling	Surface Seal	Drainage	Heater Planing (with or without overlay)	Overlay	Reconstruct	Do Nothing
Riding comfort					X	X	X	X
Fracture								
Fatigue				X		X	X	X
Temperature		X				X	X	X
Load (weight)						X	X	X
Distortion								
Traffic				X	X	X	X	X
Swelling				X		X	X	X
Settlement						X	X	X
Disintegration								
Traffic			X			X	X	X
Material durability	X						X	X
Safety								
Aggregate polishing			X		X	X	X	X
Hydroplaning			X		X	X	X	X

Figure 1. Logistic plan for proceeding from pavement network monitoring to individual project analysis.

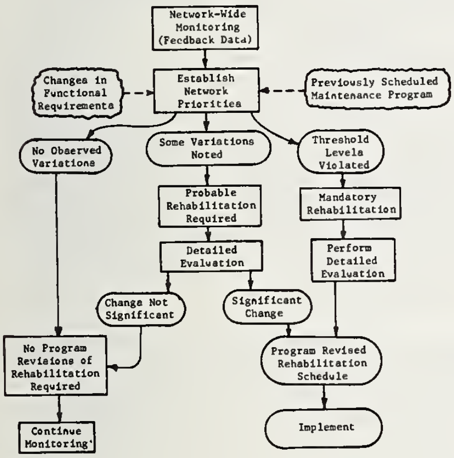


Figure 2. Serviceability-history-performance concept (3).

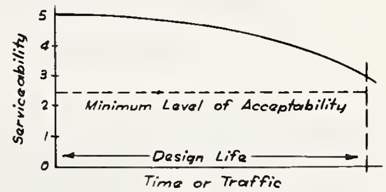
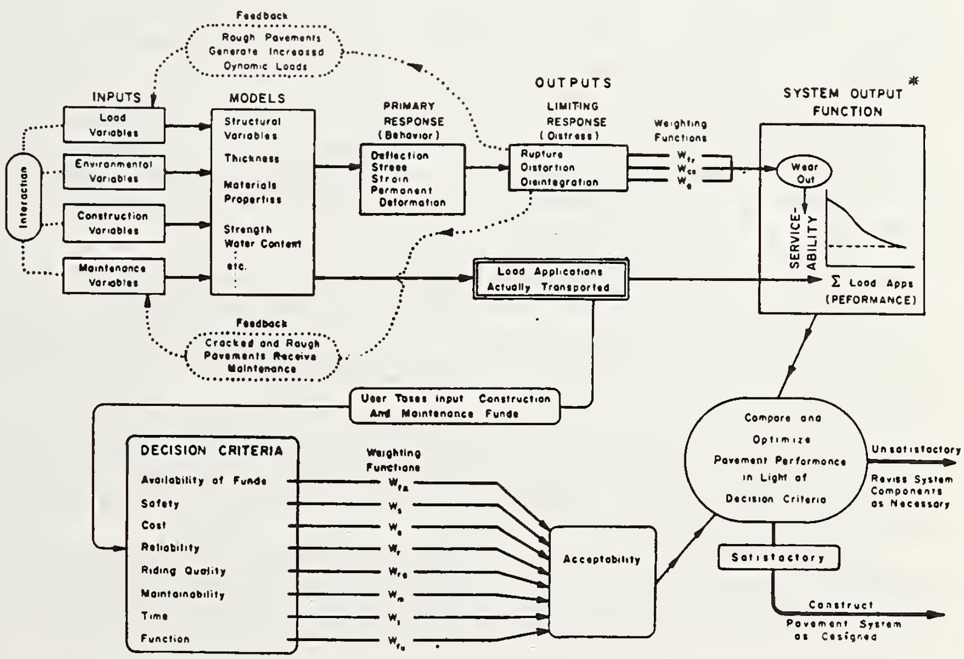


Figure 3. Conceptual pavement system (20).



sidered more important at this stage to monitor the whole system in a limited program than a small part of the system in detail.

Based on information obtained through the monitoring program, either as individual factors or as combinations of factors, a system of priorities can be established that will group pavements according to the need for major maintenance or rehabilitation. The present or future functional requirements for the pavement must, of course, be considered at this stage.

The following three general categories of priorities might be established:

1. No rehabilitation required--Monitoring indicates that the pavement is performing as programmed and that no changes need be made in the maintenance schedule during the performance period.
2. Probable rehabilitation required--Evaluation data are lower than acceptable on one or more factors, indicating that study and evaluation are needed. Based on this complete investigation, a revised rehabilitation strategy might be formulated and programmed. In some cases, the detailed evaluation might still conclude that no change is required in the maintenance schedule.
3. Mandatory rehabilitation--Some evaluation factors may require mandatory action when violated (e.g., low coefficient of friction or excessive cracking). In such cases, a detailed project analysis would be made of all evaluation categories in order to determine precisely what rehabilitation would be most effective. A revised maintenance-rehabilitation schedule would necessarily result in these cases.

REHABILITATION AS PART OF AN INTEGRATED PAVEMENT MANAGEMENT SYSTEM

Historically pavement design has been thought of as a one-shot process that could be done with complete confidence if we had rational methods. Gradually, engineers are becoming aware that this is an unrealistic definition of design. Even in complex design and manufacturing problems, the concept of quality control and setting confidence limits on the product are becoming well accepted. As a result of the AASHO Road Test (2, 3), the serviceability-performance concept for pavements became well-defined. With this concept (Fig. 2), the life cycle of a pavement is shown in terms of a serviceability time or traffic curve where the pavement is constructed at a given level of serviceability and reaches the minimum acceptable level of serviceability at the end of its design life.

Thus any pavement that will carry the expected traffic for the design period would represent an acceptable design. On the other hand any pavement that falls below the minimum acceptable level of serviceability prior to the end of the design life or analysis period would be unacceptable. Such a simplistic analysis cannot realistically consider pavement maintenance nor rehabilitation. This has been handled in the past by assuming that normal maintenance would be performed as necessary on any acceptable design.

In 1967-68 Hudson (7) extended this concept to show the pavement performance curve as a system output function for a pavement system (Fig. 3). This concept recognized maintenance as an important part of the pavement system and showed both construction and maintenance variables as input data to the system. In 1970 a team of researchers from the Texas Co-operative Highway Research Program presented a working pavement design concept that carries the rational pavement design one step further (9). It not only included maintenance as an important and necessary input variable, but recognized that most pavements are not constructed to remain smooth for the entire design life without additional work (Fig. 4). It noted that most pavement designs involve two or more performance periods in which (a) a pavement is constructed at an initial serviceability level, (b) it deteriorates to an unacceptable level at some time during its life and is rehabilitated to an acceptable level, and (c) it continues to serve traffic.

This process may be repeated several times depending on the desires of the designer, quantified in the terms of constraints and cost factors, and acceptance by the user.

Table 2 shows an example of a pavement systems analysis output from the Texas FDS method. Note that the solutions not only provide for initial construction but also program rehabilitation (overlays) and major maintenance (seal coats) and consider routine maintenance costs. This example is not intended to be complete but merely shows how one state is using a system to consider these factors.

In September, 1970, Haas and Hutchinson (6) coined the phrase management system with reference to highway pavements. As they outlined it, the pavement must be designed and the design must be communicated for implementation, constructed, maintained, monitored for feedback information, and rehabilitated as needed one or more times for the total design life (considering cost and all required inputs).

Thorough examination of actual highway pavement life histories by several state highway departments as well as on airfields by the U. S. Air Force and the U. S. Corps of Engineers indicates that this cyclic process is more realistic than the so-called one-shot design method. As a matter of fact, almost no pavements can be found that serve out a predetermined design life of 20 years

Figure 4. Working pavement system.

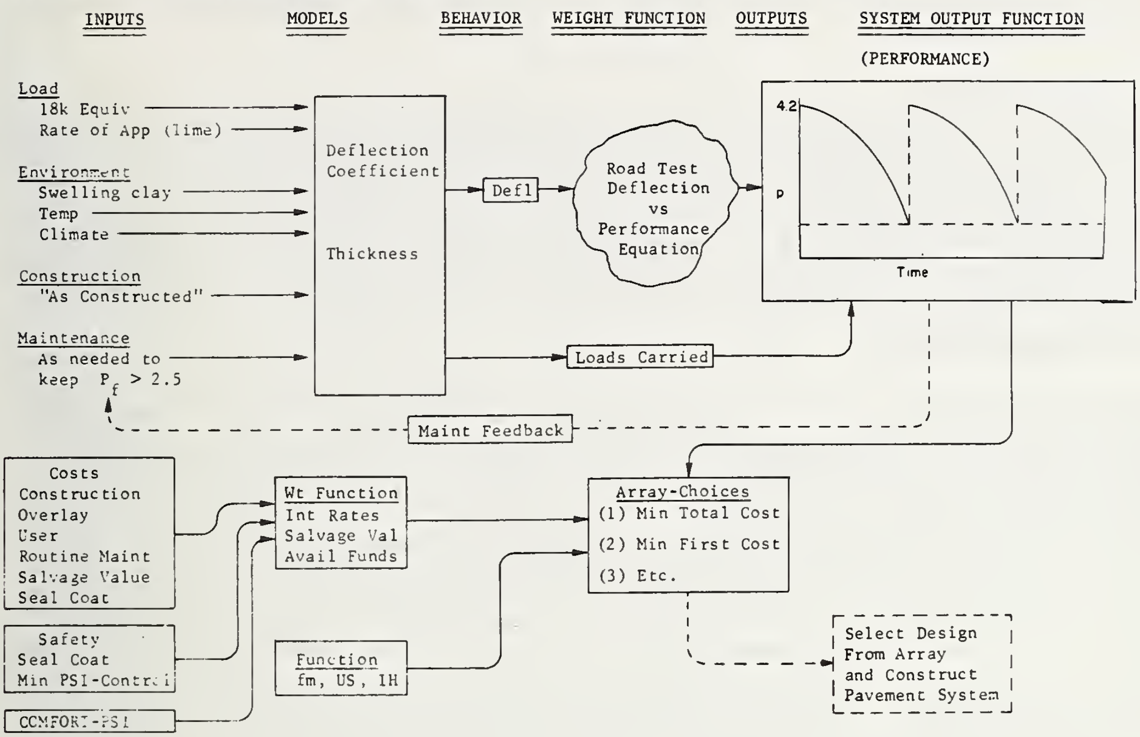


Table 2. Sample Texas pavement systems analysis illustrating output for maintenance program.

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  PROB      1                   A SAMPLE PROBLEM

FOR THE 3 LAYER DESIGN WITH THE FOLLOWING MATERIALS--

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

3    THE OPTIMAL DESIGN FOR THE MATERIALS UNDER CONSIDERATION--
     FOR INITIAL CONSTRUCTION THE DEPTHS SHOULD BE
          ASPHALTIC CONCRETE        1.00 INCHES
          CR. LIMESTONE-I           8.50 INCHES
          GRAVEL-1                  6.50 INCHES

     THE SCI OF THE INITIAL STRUCTURE  =   .521
     THE LIFE OF THE INITIAL STRUCTURE =  4.91 YEARS
---> THE OVERLAY SCHEDULE IS
          2.00 INCH(ES) (INCLUDING 1 INCH LEVEL-UP) AFTER  4.91 YEARS.
          1.50 INCH(ES) (INCLUDING 1 INCH LEVEL-UP) AFTER  9.95 YEARS.
          1.50 INCH(ES) (INCLUDING 1 INCH LEVEL-UP) AFTER 16.20 YEARS.
          TOTAL LIFE =       23.81 YEARS

---> SEAL COATS SHOULD OCCUR AFTER
          (1)    9.91 YEARS
          (2)   14.95 YEARS

     THE TOTAL COSTS PER SQ. YD. FOR THESE CONSIDERATIONS ARE
          INITIAL CONSTRUCTION COST                2.000
   ---->  TOTAL ROUTINE MAINTENANCE COST            .166
          TOTAL OVERLAY CONSTRUCTION COST           .882
          TOTAL USER COST DURING
                 OVERLAY CONSTRUCTION               .203
          TOTAL SEAL COAT COST                      .233
          SALVAGE VALUE                            -.679
          TOTAL OVERALL COST                       2.804

     NUMBER OF FEASIBLE SESIGNS EXAMINED FOR THIS SET --       145

       AT THE OPTIMAL SOLUTION.THE FOLLOWING
       BOUNDARY RESTRICTIONS ARE ACTIVE--
                1. THE MINIMUM DEPTH OF LAYER  I
                2. THE MINIMUM TIME BETWEEN OVERLAYS
```

or more without some maintenance or rehabilitation. At the recent HRB workshop on pavement systems (17) these concepts were thoroughly discussed and generally accepted by a wide variety of pavement design and research engineers.

Changes in Input Variables

The concept of continued monitoring or reevaluation becomes even more realistic when we realize that a change in any of the key input variables from the estimates in the original design problem can result in a significantly altered performance history for the pavement. For example, if the traffic or average axle load on a highway is significantly increased over the estimate because of the development of a new industrial park, the estimated pavement life can be shortened markedly. Likewise, if the environmental conditions are considerably better than predicted, the pavement may last longer than expected. If worse moisture conditions prevail, a shorter life could result.

Optimization of Pavement Design

Building on the concepts outlined above, the pavement management concept consists of several cycles in which the pavement system is repeatedly analyzed and examined. The first of these might be called the design cycle. In general terms, subsequent cycles could meaningfully be called rehabilitation cycles. The initial or design cycle of the process involves the selection of the optimum performance history for the initial structural section based on the input data and imposed constraints of the problem as shown in Fig. 5.

As a part of this process, some initial set of materials and geometry is selected for construction. For simplicity, let us assume now that an example pavement is constructed as required in the initial design cycle and is projected to have two performance periods. Figure 6 illustrates the projected performance history of the pavement under consideration. From this figure we see that the first performance period is expected to last 10 years. Rehabilitation is to be performed at that time, and a second performance period will last at least beyond the selected 20 year design life. In this case it will be necessary at 10 years to perform some sort of rehabilitation, presumably an overlay as projected during the initial design cycle, to strengthen the pavement and restore riding quality to its initial level. If our original analysis were exact, the second performance period would also obtain as projected.

Analysis for Rehabilitation Cycle

In reality the probability of such exactitude in the systems models is extremely unlikely. Many changes can and will occur. Thus it will be highly desirable to evaluate the pavement periodically during each performance period. Typically this should be done annually or biannually depending on other factors. However, at least 1 year prior to a projected overlay, the actual performance history, serviceability index, and physical characteristics of the pavements should be monitored and carefully evaluated in order to provide a status report on the physical condition of the pavement section. The engineer should carefully compare reported data with selected decision criteria.

The evaluation of the pavement can and should include condition surveys, serviceability or roughness evaluation, mechanistic evaluation, and skid resistance measurements. Based on the information from these monitored data it is possible for the pavement engineer to evaluate the pavement system with the new input data in what might be called a second design cycle or a rehabilitation cycle. In this case, the same performance prediction models, decision criteria, and constraints are placed on the process as for the initial design. As illustrated by Texas, Utah, and others (9, 18, 19) the same computer program can actually be used for this process.

Reanalysis at Any Time

Now that we have seen the concept of a rehabilitation analysis cycle when required by the original design predictions, we may expand the concept to show that it is possible to run this reanalysis at any time the pavement engineer chooses. Thus, if an unexpected increase in traffic or change in other factors becomes evident and it is desired to strengthen the pavement, it is possible to rerun the analysis package with input data including the projected traffic and the existing condition of the pavement as obtained from the appropriate monitoring methods. In some cases, unexpectedly good or poor performance of the pavement materials will make it desirable to run a reanalysis cycle even though the pavement may not have reached its minimum serviceability level.

To generalize, we may think of the pavement as a system that is functioning all the time. We may examine that system at any time with available evaluation techniques and the system analysis computer program. To expand the concept, the pavement evaluation techniques of all types become the monitoring methods for the pavement system.

It should be understood that it is possible for excessive distress to trigger rehabilitation action to reduce the distress and prevent the expected degradation of PSI values.

COSTS AND ECONOMICS

The entire pavement design and management concept is related to costs and economics. In the pavement management process, the costs that must be considered include not only initial costs, but also routine maintenance costs, rehabilitation costs, and so-called user costs. User costs are those costs that the pavement user pays, both directly and indirectly with relation to the pavement facility or lack of it. User costs are primarily related to a pavement in poor condition that results in excessive roughness, perhaps loss in speed, and vibration damage to the vehicle. A second major user cost is related to the detour and delay cost that the user suffers with relation to time required for maintenance and rehabilitation of a given facility. Considering these costs, along with the initial construction costs and the time value of the money, makes it possible to evaluate true relative costs of various rehabilitation pavement strategies and to select one that is optimum based on the decision criteria selected.

Timing

The second major aspect of economics requires that there be good coordination between the various types of maintenance and rehabilitation. Thus, if a pavement indicates inadequate skid resistance and a skid resistant seal coat seems warranted, it should not be applied until a structural evaluation determines that a structural overlay is not also required. Without this coordination, the seal coat might be applied one year, and then a major reconstruction or overlay required the next year would completely destroy the value of the prior seal coat.

Excessive Costs Criterion

A final important aspect of economy is the concept of excessive maintenance costs. We normally portray a serviceability-time history function as shown in Fig. 2. However, it is possible to extend the life of a pavement suffering severe distress by providing extensive heavy maintenance. This is sometimes done when a major highway is suffering damage but inadequate funds are available for rehabilitation. Fig. 7 illustrates such a concept where a pavement is very near its unacceptable level but remains slightly above it because of the amount of the maintenance expended. This is shown in illustration by an accumulative maintenance cost curve plotted on the same figure.

Thus, a pavement might be considered to be failed when it reaches an unacceptable level of serviceability or when the cost of maintenance becomes excessively high. This might be controlled in an actual field situation by programming maintenance funds according to a predetermined acceptable formula. When it becomes impossible to maintain the road adequately for the formula funds, then a detailed evaluation should be performed.

PREDICTION MODELS

To anticipate the need for rehabilitation on a project basis or to project program funding for a highway network requires that the performance, condition, or behavior at some time in the future can be predicted. Very few agencies have attempted to obtain the type of information needed to establish prediction models. The AASHO Road Test staff developed empirical (statistical) models to represent the serviceability history of test sections included on the project. Washington and Utah have each developed general methods for projecting performance trends that can serve as initial models in making predictions.

Prediction models can be developed by correlations with observations and measurements (empirical). This approach, used by the AASHO Road Test staff, depends on the ability to find some combination of factors (e.g., structural geometry, wheel loads, load repetitions, etc.) that can be statistically correlated with the dependent variable.

Prediction models can also be developed by a mechanistic approach. This method can potentially combine pavement geometry, material properties, load, and certain environmental parameters into a more reliable predictive equation. However, it should not be concluded that mechanistic models can be related to performance without some additional research or empirical data. Mechanistic models predict behavior or distress. There still exists the very difficult problem of relating distress to performance. In the Austin Workshop on Pavement Systems, this was listed as the number one research problem in rational pavement design. The problem is also paramount in pavement rehabilitation methods. In fact, the same predictive models that are applicable for pavement design must also be used for subsequent redesign or rehabilitation. In all probability the accumulation feedback data will indicate that some coefficients in the model will change after rehabilitation even though the model may contain the same terms. It is essential to have these prediction models for both the original design cycle and the rehabilitation cycle of the pavement.

Variability

Because most of the elements of a pavement response system are stochastic, it should be apparent that predictions can be made only within certain levels of probability of being correct or within possible confidence intervals. Precise estimates or predictions are not possible, and as the prediction period becomes larger, it can be expected that the level of confidence for performance predictions will decrease. To minimize possible errors in fund programming based on future predictions, it is necessary to monitor continuously the various factors pertinent to the establishment of rehabilitation priorities and strategies. Based on such monitoring, dynamic programming procedures can be used to modify future maintenance plans and programs. Thus, the method for predicting rehabilitation must be designed to be sufficiently flexible to allow for change as more and more information is collected.

DETAILED PROJECT ANALYSIS

Looking at the pavement management systems in Figures 3 and 4, we recall several major variables and subsystems, including inputs, prediction models, safety measurements such as skid resistance, costs, behavior, distress, performance, and decision criteria. In order to relate the system to rehabilitation a simplified version is given in Fig. 8. To reiterate the process, a variety of physical information is required as input data for a series of predictive models. These predictive models are based on the theories or the empirical relationships that have been developed for pavements behavior. For rational pavement design, these models must include at least (a) a model or models related to skid resistance and safety; (b) cost models including initial construction, maintenance, and user cost; and (c) performance prediction models that relate behavior, distress, and performance. All rational pavement management systems must consider all these factors. We may divide the diagram of the pavement management system into an upper and lower half, the upper half including predictive models along with inputs and outputs and the lower half involving decision models. Fig. 9 shows simplified relationships among the seven aspects of the predictive portion of the system. In addition, the figure illustrates the six major categories of possible evaluation that exist within the pavement system. A great deal of misunderstanding arises when we confuse monitoring techniques and evaluation as applied to these categories of variables.

Input Variables

In this case, inputs are related directly to the pavement, including the physical structure and material strengths. These can be monitored by physical testing and sampling to provide direct information about layer thicknesses and material properties. Load testing of the total pavement structure is not included here because it involves the pavement response or behavior and not the input information itself (see behavior below).

Skid Resistance --Skid resistance could be categorized in a general response system, however, it is amenable to treatment and evaluation alone and is normally handled by measuring the coefficient of friction on the pavement surface.

Cost --Cost can only be monitored if cumulative records of expenditures including initial construction costs and subsequent maintenance expenditures are compiled. Most highway agencies do maintain information along these lines; however, attention is needed to ensure that costs associated with maintenance of the pavement are adequately separated from costs associated with mowing, garbage pickup, and other routine maintenance duties.

Behavior --We have defined behavior as the immediate response of the pavement to load. Thus, load-deflection testing of all types including (a) plate load tests, (b) Benkleman Beam measurements, and (c) dynamic deflections fall into this category. Although information about the physical structure of the pavement is often inferred from behavioral evaluations, it should be remembered that these load testing techniques evaluate only the behavioral response of the pavement, not the physical properties directly.

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

Performance --A final category of pavement evaluation of major interest is pavement performance. Because performance is the accumulated serviceability history of the pavement, its evaluation implies a time-related accumulation of data. This is best accomplished by evaluation of the serviceability level of the pavement periodically (annually) with a pavement serviceability rating or profile evaluation that yields an index of serviceability. At the very minimum, realistic evaluation of performance requires two pavement serviceability estimates, one at the time of evaluation and a previous one usually at the time of construction. A single serviceability measurement at the current time can tell the pavement engineer whether or not the pavement is below the currently desirable level. The change in serviceability from some prior time or rate of change of SL (ΔSL) provides information about the performance history.

Interaction --The six separate pavement concepts outlined above must not be confused or used interchangeably. For example, the fact that some people evaluate serviceability level by using a serviceability equation that involves cracking and patching does not mean at all that the resulting equation provides an adequate evaluation of distress and performance. The serviceability equation is primarily a performance equation tool.

Likewise, the fact that dynamic deflection measurements may be used to estimate pavement structural thicknesses and properties should not confuse the user. These behavioral measurements can be used to estimate inputs only in conjunction with some type of theory or model. The structural input values themselves can only be directly evaluated with a destructive test or sampling procedure.

Evaluation Decision Criteria

The pavement evaluation process involves decision criteria related to threshold levels of each of the monitoring techniques. In some cases the threshold levels are absolute in themselves, providing information on which required action can be based. A minimum acceptable level of serviceability is an example of such an absolute threshold where a design agency policy might establish a minimal level of 2.5 for Interstate highways. Pavement sections dropping below this level would then require some type of rehabilitation.

Other types of decision criteria may be more flexible, either to serve as indicators to the engineer or to be varied with a particular pavement design as needed. Thus, in the first instance, the amount of cracking in a particular pavement section may have no absolute limit, but as cracking progresses it can be of significant concern to the engineer and indicate to him that rapid changes are taking place and subsequent evaluation is needed. In the case of adjustable criteria, a particular pavement design method might predict an acceptable deflection of 0.015 inch for a particular section. Assume that subsequent evaluation indicates a deflection significantly above 0.015 inch. This indicates to the pavement engineer that the pavement is behaving differently than expected, and thus a more complete evaluation and/or analysis might be justified. In each case, three possible paths may be followed: (a) continue maintenance for predicted life as programmed, (b) call for a more complete evaluation and analysis of the pavement section to provide a better basis for decision, or (c) take immediate appropriate rehabilitation action as required based on the violated criteria.

Combinational Decision

Certain highway agencies have established pavement rating methods that combine one or more of the evaluation techniques outlined above into a single index. Such combinational decisions usually involve assigning a weighting factor to condition survey information and combining it with the serviceability level. Whereas such combined indexes can prove very useful in analyzing various parts of a pavement network and in providing a simple method of ranking pavements into relative condition categories, combining the information is not very useful for evaluating and analyzing a specific pavement section for rehabilitation decisions. There is nothing wrong with combining indexes if the engineer finds the combined index useful. On the other hand, each of the evaluation techniques has a specific use of its own, and these can become somewhat clouded when combined.

Relations Between Evaluation Techniques

Some confusion has been generated over the past decade concerning serviceability evaluations. Some people assume that serviceability estimates are intended to replace condition surveys or mechanistic evaluations. Because the AASHO Road Test PSI equation contains a small multiplier for cracking and patching, people somehow assume that the pavement condition variables have been combined with serviceability estimates, but that inadequate weight has been assigned. These assumptions are incorrect in both cases. It should not be suggested in any way that the serviceability evaluation is to replace the condition survey or distress evaluation. Quite the contrary, the distress evaluation is an extremely good indicator of future changes in performance. As a pavement begins to behave badly and reach some limiting level of behavior, it will begin

Figure 5. Multiple pavement strategies to be analyzed in initial or design cycle of management process.

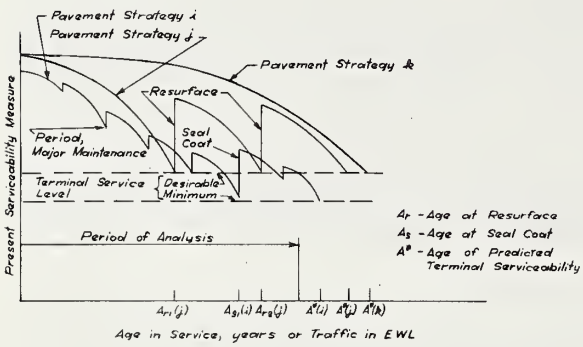


Figure 6. Performance history plot.

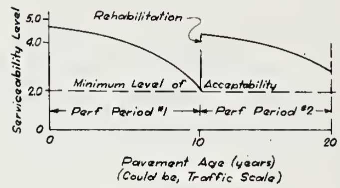


Figure 7. Example pavement record.

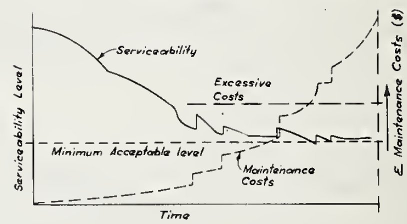


Figure 8. Simplified pavement system.

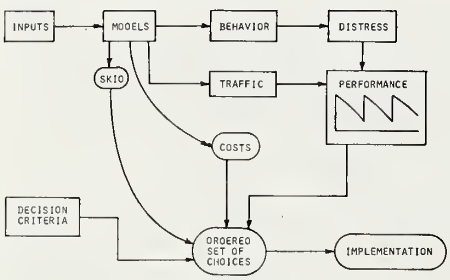


Figure 9. Simplified predictive portion of pavement system analysis relating six methods of prediction.

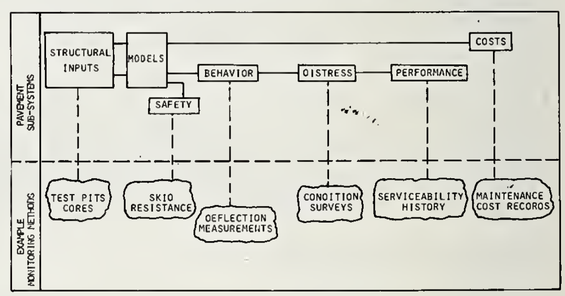
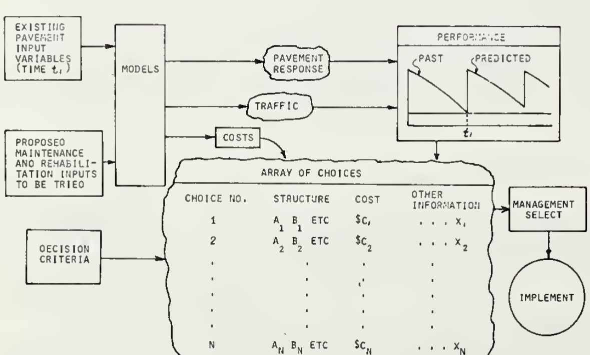


Figure 10. Conceptual pavement system structured especially for pavement rehabilitation process.



to crack or permanently deform and its distress index will increase. Such cracking may not result in an immediate increase in roughness or decrease in serviceability. More likely, the increased roughness will follow or lag behind the cracking by some period of time. In many cases, a decision to overlay or rehabilitate the pavement will be made on the basis of distress evaluation rather than serviceability evaluation. This is a perfectly valid, rational situation where time and funds allow such a decision.

If a pavement reaches an unacceptably low serviceability level with or without cracking because maintenance funds were insufficient or because of an unusually severe environmental condition (hard winter), then the serviceability evaluation may dictate that rehabilitation be performed. This may occur in the absence of cracking as in the case of swelling soils.

Individual Pavement Sections

It is quite common for highway engineers to think of pavement sections several miles in length. These are usually related to a particular control section and job identification. Although these sections are quite useful for many identification purposes, there are usually a number of physical changes in the terrain, the materials, the traffic, or other factors that vary from point to point within the limits of the job. For this reason, it is practically impossible to consider a long segment of pavement to be uniform; therefore, subdivision into feasible section size is essential. On the other hand, physical limitations of measurement and construction do not make it possible to consider extremely short lengths of pavement, say 100 to 200 feet, as being different from adjacent sections. This breakdown could lead to an undesirably large number of analysis sections to deal with that could not be handled economically by available analysis methods. This problem has been handled in a reasonable way by several agencies including King County, California, the Texas Highway Department, and others. In general, these agencies make a compromise that involves dividing the pavement into quarter-mile long sections. Each quarter-mile is considered to be a uniform pavement section, and variables within this section are combined and then specified with a mean and a variance. If adjacent sections are very similar, they may be combined for convenience or left separate. This sectioning of pavement jobs has proved to be an effective technique in practice and is recommended for future use.

IMPLEMENTATION OF REHABILITATION ANALYSIS

Once a decision is made to rehabilitate the pavement in question, the next step is to obtain current data (if not already available) on the existing structural condition of the pavement. In other words, all input variables must be evaluated to run a reanalysis of the pavement system package. Many design agencies do not attempt a complete reanalysis for overlay design; however, any realistic method must involve evaluation that provides some kind of performance prediction model.

In its desirable form, the process is shown in Figure 10 in which several alternative overlay designs are compared, along with associated costs and resulting benefits.

A decision must be made from the available possibilities and this decision implemented. The designer must prepare a set of documents to transmit the rehabilitation decisions to the group charged with construction or maintenance. These documents are usually plans, specifications, and bid documents. Actual work or implementation may be by contract or agency labor. To finalize implementation, a series of as-constructed evaluations should be recorded for future use and analysis. These data may result from construction acceptance testing or special evaluations.

APPLICATION OF THE FRAMEWORK

One of the most important considerations is the need to have systematic procedures for monitoring the pavement. Specifically, it will be important to have definite procedures for measuring (a) the condition, (b) serviceability, and (c) behavior of the pavement. These determinations need to be made by procedures that are realistic; that is, procedures that are accurate but not excessively expensive, can be accomplished periodically, and will provide sufficient information for the applicable evaluation procedure. A great deal of work has been done to codify methods for skid resistance and surface roughness and serviceability (24).

In evaluation, it will be necessary to conduct detailed studies, with appropriate analysis, in order to attempt to optimize the rehabilitation decision. Detailed methods for measuring both the physical properties (geometry, material properties) and behavior characteristics (deflection, curvature, strain) of the pavement will be necessary. These measurements need to be made in a manner compatible with the planned method of analysis. For example, if a layered elastic system is to be used, it will be necessary to test materials to determine their elastic constants. If empirical structural design procedures are used, the appropriate test value (CBR, R-value) will be required as input to estimate the adequacy of a given pavement section.

To make a decision for rehabilitation requires a planned procedure or strategy by which specific rehabilitation objectives are met. That is, guidelines for decisions are needed in order that the total process can be developed.

In even the best of plans, there are always exceptions that must be provided for by contingency planning. In the problem of rehabilitation, there are special cases that require special treatment. Reflection cracking is such a case; thermal cracking might be another. Some special information needs to be developed to provide engineering criteria for the treatment of these problems.

Finally, it would be a mistake to assemble a group of prestigious engineers to discuss the real-world problems and solutions without also discussing ideal procedures and solutions. Thus, an important charge to the Workshop is to put aside the encumberances of knowing how difficult the problem is and to bring fresh, new approaches to the problem of pavement rehabilitation in an idealistic framework.

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APPENDIX

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. As such, it is a direct function of the present serviceability history of the pavement.
2. Serviceability is the ability of a specific section of pavement to serve traffic in its existing condition.
3. Behavior is the immediate reaction or response of a pavement to load, environment, and other inputs. Such response is usually a function of the mechanical state, i.e., stress, strain, or deflection, which occurs in response to the input.
4. Distress mechanisms are those responses that can lead to some form of distress when carried to a limit, e.g., deflection under load. Some behavioral responses may not provide distress mechanisms.
5. Distress is the visible consequence of various mechanisms of distress that usually lead to a reduction in serviceability.
6. A system is something that accomplishes an operational process. That which is operated on is usually input, that which is produced is called output, and the operating entity is called the system. The system is a device, procedure, or scheme that behaves according to some description, its function being to operate on information, energy, and/or matter in a time reference to yield information, energy, matter and/or service (Ellis and Ludwig).
7. Systems failure may be expressed as a condition in which the combined distress in the system response has exceeded an acceptable level based on the decision criteria as when the serviceability level drops below an acceptable level.
8. Model is a system of postulates, data, and inferences presented as a mathematical description of a conceptual reality.
9. Feedback is the collection and reversion of the pavement distress or limiting response data to the data bank for use in analysis, maintenance studies, rehabilitation scheduling, and so forth.
10. Monitoring, in the context of this report, is the process of sampling response data of a pavement such as coefficient of friction, serviceability, or deflection.
11. Evaluation involves the monitoring of the pavement and also includes the incorporation of such information in a prescribed analysis method or procedure.
12. Rehabilitation is the act of restoring something to its former condition or, for a pavement, restoring its ability to fulfill its function adequately.
13. Maintenance is the act of keeping something in its present condition.
14. Minor maintenance is that maintenance considered as a matter of routine by field forces on a programmed, budgeted basis.
15. Major maintenance is that maintenance requiring special engineering, management, and budget consideration.
16. Pavement management is that process of designing, communicating design, implementing, constructing, maintaining, monitoring, and rehabilitating a pavement section to provide for the required performance.
17. Rehabilitation strategy is the complete set of activities and decisions that are required to make up one rehabilitation action (e.g., monitor, analyze, choice, implement).

SURFACE EVALUATION OF PAVEMENTS: STATE OF THE ART

Ralph Haas

INTRODUCTION

Scope

Expenditures on pavement rehabilitation have increased significantly for most highway and airport agencies during recent years. A major reason is that the network of post-war pavement construction is now requiring more intensive maintenance and much of it is in imminent need of overlay repairs. These requirements are being compounded by the effects of continually increasing traffic volumes and loads.

Pavement designers have directed most of their research and development efforts to new pavements. There is now an additional need to extend design technology to the area of pavement rehabilitation. In order to do this, it is necessary not only to have the appropriate models for analysis but also to provide the methodology for evaluating existing pavements.

The evaluation of existing pavements for the purpose of assessing rehabilitation needs can be in terms of

1. Evaluation of structural capacity,
2. Evaluation of physical deterioration, such as cracking, deformation, and disintegration,
3. Evaluation of user-related effects such as roughness or serviceability, safety, and appearance,
4. Evaluation of user-related costs and benefits associated with varying serviceability and safety and with various rehabilitation measures.

These are all closely related. This paper considers mainly the methodology for evaluation of physical deterioration and for the evaluation of user-related effects. Papers by McComb (1) and Witczak (2) are directed to the first aspect of structural evaluation, while the latter aspect of user costs and benefits is briefly mentioned in this paper.

Objectives

The general purpose of this paper is to document the current methodology for acquiring and using pavement evaluation data. More specifically, the objectives are:

1. To define briefly the role of pavement surface evaluation in the broader context of pavement management,
2. To present the fundamental concepts underlying the various pavement evaluation measures which are user and manager oriented,
3. To present the existing state of the art for measuring surface roughness and serviceability, surface distress or condition, safety and user costs associated with surface characteristics,
4. To outline various approaches for interpreting measurement data and to consider the uses and implications of the results.

PAVEMENT EVALUATION AND REHABILITATION AS A PART OF PAVEMENT MANAGEMENT

A pavement management system consists of a comprehensive set of activities that go into the planning, design, construction, maintenance, evaluation, and research of pavements. The basic concepts have been presented (1 - 9), and several working systems have been implemented (8, 10, 11, 12). These explicitly recognize that pavement rehabilitation needs, and the evaluations or estimates on which they are based, should be integral parts of the pavement management system. It is useful to this paper to outline briefly the role of surface evaluation within such a pavement management system. A more comprehensive treatment of the subject area is contained in a paper by Hudson and Finn (13).

Figure 1 is a flow chart of the foregoing major classes of activities that compose a pavement management system. It indicates that pavement evaluation provides the information to the planning phase for assessing deficiencies on a network basis, including current and future rehabilitation needs. These needs are associated with existing in-service pavements. But what about future rehabilitation needs for new pavements that are only at the planning and design stage?

Figure 1. Major classes of activities in a pavement management system.

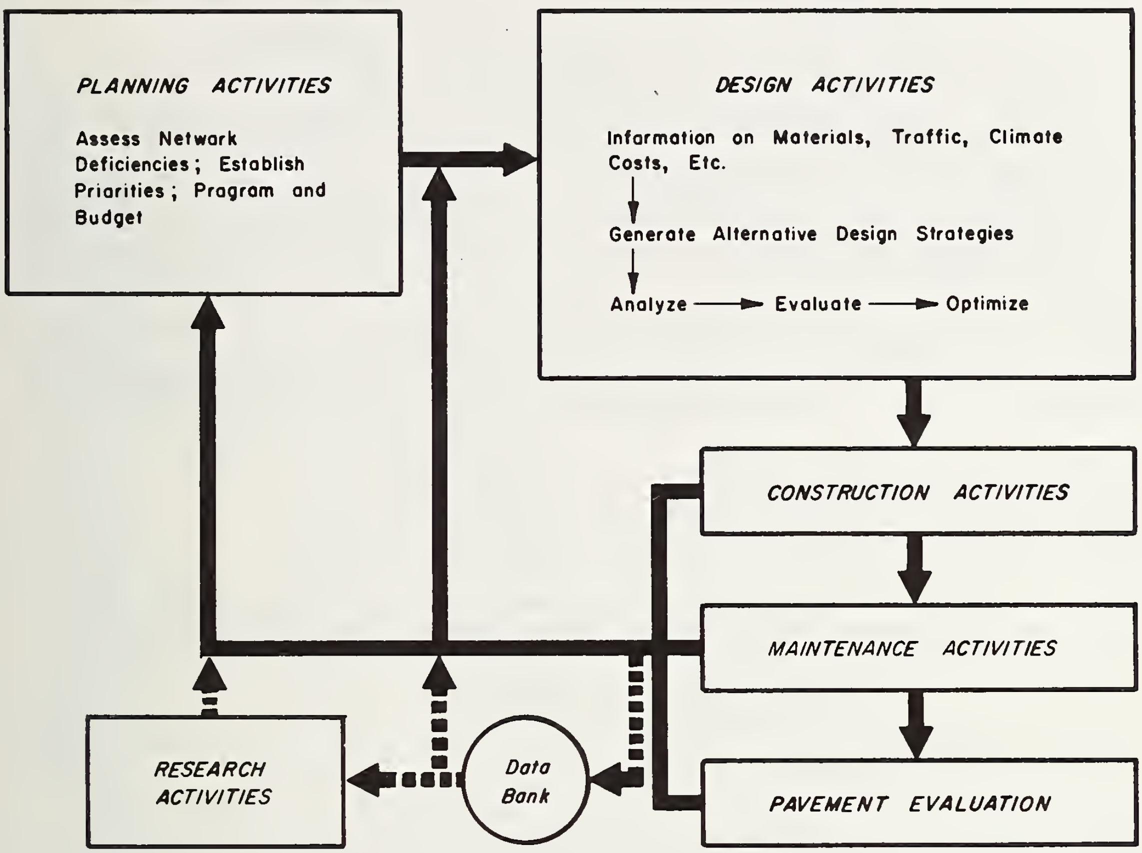
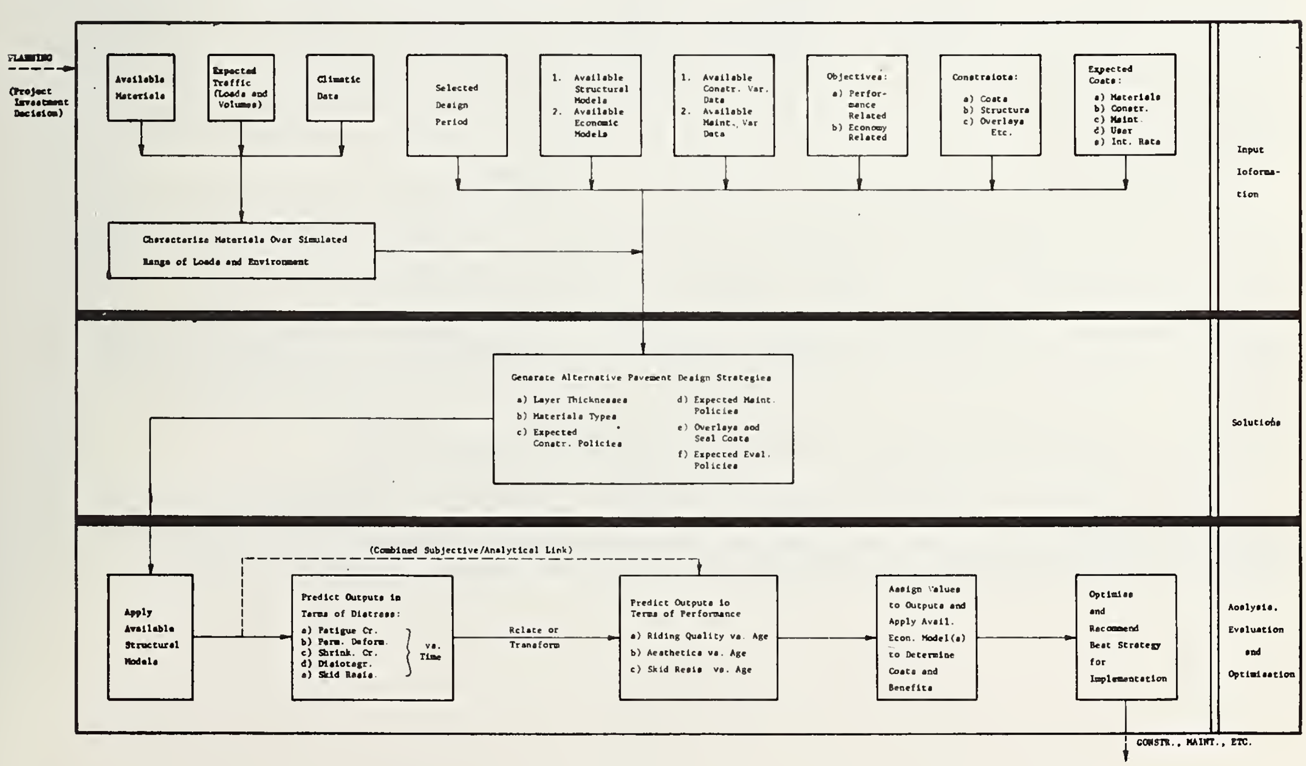


Figure 2. Major pavement design activities.



It has been suggested that, in the case of these new pavements, the designer must generate rehabilitation alternatives as part of his overall design strategy (9, 14, 15). He then analyzes these alternative strategies to predict their outputs and evaluates them to select the best one. Figure 2, which is an expansion of the design phase of Fig. 1, illustrates the concept. The top row shows the information either available to or acquired by the designer. The middle row lists the key components involved in generating alternative design strategies, while the bottom row shows the analysis and evaluation performed on these strategies. This latter row of activities is somewhat idealized in that the technology for relating predicted distress to predicted performance requires considerably more development. In fact, the 1970 HRB Workshop considered this as a number one research priority (16).

The generation of alternative design strategies should include, as shown in Fig. 2, future rehabilitation alternatives in terms of overlays, seal coats, and expected maintenance policies. When the designer then conducts his analysis and evaluation, he must include in his predictions the effects of these rehabilitation alternatives. Two of the major outputs that he must predict, over the analysis period, are performance (i.e., serviceability or riding comfort vs. age history) and safety (i.e., skid resistance). These are illustrated in Figure 3 for two alternative design strategies, i and j, together with the economic implications of these strategies. Rehabilitation, in terms of major maintenance and resurfacing, is included in these two strategies.

It is the function of pavement evaluation to measure periodically these and other outputs in order to

1. Provide data for checking and updating the design predictions,
2. Reschedule rehabilitation measures as indicated by updated predictions,
3. Provide data for upgrading design models, and
4. Provide information for updating the network rehabilitation programs.

Pavement evaluation thus serves both planning and design activities of pavement management. It is a key part of the pavement management system and provides the means for assessing rehabilitation needs on both a project and a network basis.

PAVEMENT PERFORMANCE EVALUATION

Basic Concepts of Pavement Serviceability and Performance

The primary operating characteristic of a pavement is the level of service provided to the users. Variation in serviceability provides a measure of pavement performance. The best known definitions and procedures for measuring serviceability in North America are those developed at the AASHO Road Test, (17) and in the Canadian pavement evaluation studies (3, 14, 18-21).

These serviceability measures are supposed to simulate the user's opinion or evaluation of the riding quality provided by the pavement. In the AASHO and Canadian studies, procedures for obtaining the user's opinions were developed by forming rating panels and having the members of these panels drive over a number of pavement sections. Certain ground rules were established for these rating sessions (17, 18, 21). Each member records his independent, subjective opinion on the type of form shown in Fig. 4. The AASHO terminology for each such rating is individual present serviceability rating, with the mean of the individual ratings termed 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 the evaluation of pavement riding quality only (3).

The major difference between the two approaches, as shown in Fig. 4a to 4b, 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 five. As well, the RCI method emphasizes 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.

Basic Differences Between Performance Evaluation and Mechanistic Evaluation

It is important to recognize the basic differences between mechanistic evaluation of pavement surfaces and user-oriented performance evaluation. Mechanistic evaluation is concerned with measuring in quantitative terms items such as structural capacity, distress (i.e., cracking, distortion or permanent deformation, disintegration), roughness, skid resistance, and more.

Figure 3. Some major pavement outputs and associated value implications.

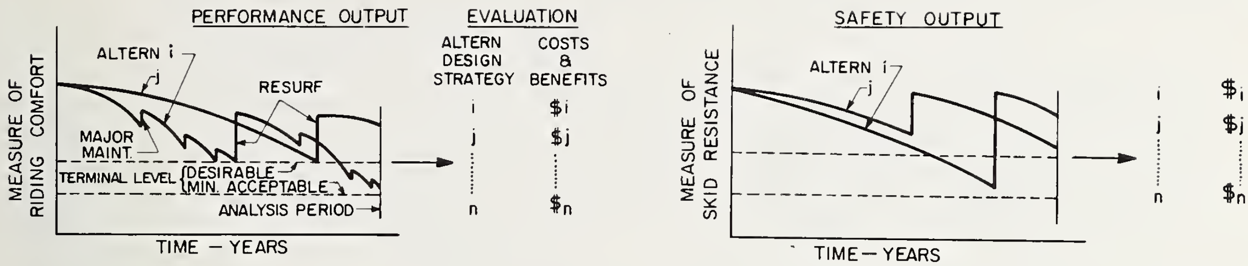
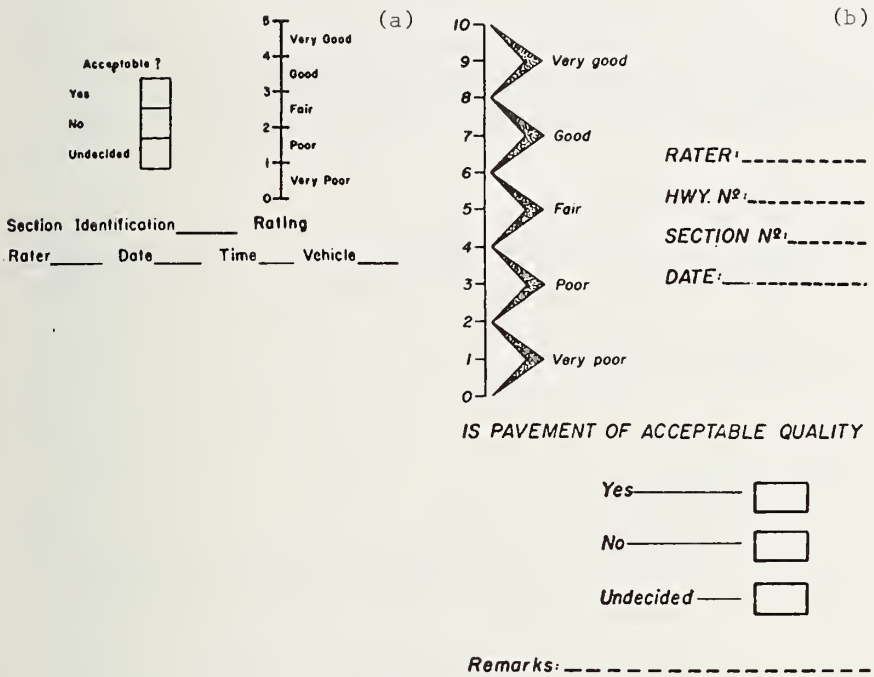


Figure 4. Evaluation forms for (a) individual PSR as used at AASHO Road Test and (b) present performance rating (now riding comfort index) as developed by Canadian Good Roads Association.



Two measures, roughness and skid resistance, can be directly related to the user. However, in the area of distress measurements (which are often referred to as condition surveys), a major source of confusion in terminilogy and concepts arises. The difference between mechanistic evaluation of distress and performance can be illustrated with a simple example. Consider a crack that occurs in the pavement surface today. It will likely have no immediate effect on how well the pavement is serving traffic. However, it is a warning of impending, accelerated future loss of serviceability. Whether it is PSR or RCI, serviceability is supposed to reflect riding quality only at the particular time measured; past or suspected future riding quality is not taken into account. Thus, the crack that occurs today is not taken into account in today's measurement of PSR or RCI, although it may well significantly affect these rating tools in the near future.

There have been attempts to provide clear definitions for the preceding terms and concepts (17, 18, 21, 22, 23, 24). Unfortunately, there are still too many instances where the terminology is loosely used; for example, referring to deflections (which really represent an evaluation of structural capacity) as performance, or using the term failure with no clear indication of whether it refers to some form of distress or some minimum acceptable level of PSR. More rigorous adherence to the proper terminology in future pavement literature would be most desirable.

Finally, it should be noted that in some condition survey methods, the results are translated, through arbitrarily assigned weighting factors, into index numbers which are then used to establish rehabilitation needs. They should not, however, be confused with performance as defined at AASHO and in the Canadian studies.

Pavement Roughness Evaluation

Pavement roughness evaluation has received considerable attention from most highway and airport agencies in North America. Roughness is the primary component of serviceability, and a large number of roughness measures have been correlated with panel ratings of PSR or RCI. Some of the more widely used methods of measuring roughness, correlating measurements, and applying the results are outlined below.

Methods of Measuring Roughness and Their Major Features

There are a variety of methods for measuring roughness currently in use on highways and airports in North America. These range from the simple to the sophisticated and include the following:

1. U. S. Bureau of Public Roads roughometer (BPR)
2. CHLOE profilometer (CHLOE)
3. Rolling straightedge (RSE)
4. British Road Research Laboratory profilometer (RRL)
5. Surface dynamics profilometer (SDP)
6. Car road meter (i.e., PCA or Mays) (CRM)
7. Precise levelling method for profile determination (LEVEL)

A considerable number of other devices have also been used. An excellent summary of these, up to 1960, has been provided by Hveem (25). In 1968, a state-of-the-art review by an HRB subcommittee of pavement condition evaluation considered some of the newer or more commonly used roughness measuring equipment, including the BPR, CHLOE, SDP, and RSE types (26). A 1969 effort by Phillips and Swift (27) compared the CRM with the CHLOE and BPR devices, and provided an excellent tabular summary of their operational features. The operating features of the other devices listed are also well documented in the literature. The following paragraphs provide only a very brief review of the essential characteristics of the various methods.

The BPR roughometer is perhaps one of the best known devices in the highway industry. Its basic form was developed in the 1920's and, as reported in Public Roads, was converted to the trailer type of device currently used by a large number of agencies. The roughometer essentially simulates one wheel of a passenger car and is comprised of a mass, spring, and damper combination. Displacement of the wheel with respect to the mass, at 20 mph, is recorded by an integrator (arranged to integrate only in one direction) coupled to an electric counter calibrated to record inches of vertical movement of the axle relative to the top of the suspension system. Accumulation of the displacement over a distance interval is called roughness index, with units of inches per mile.

The BPR roughometer has several serious limitations, including low operating speed, attenuation of wavelengths in the ride frequency range (26), and difficulties with repeatability and constancy related to calibration.

The CHLOE profilometer has also been widely used in North America. It was developed at the AASHO Road Test as a simplified modification of the AASHO slope profilometer (28). Pavement roughness is measured by the change in angle between two reference lines, one of which is determined by two small slope wheels and the other which is determined by a 20-foot frame member supported by two large rear wheels and a trailer hitch on the front. The measured slopes are reduced to a single statistic termed slope variance. A major asset of the CHLOE profilometer is good repeatability. However, it also has some serious limitations, including a very slow operating speed, inaccurate measures of wavelengths shorter than the distance between the two wheels, and lack of information on longer wavelengths.

The rolling straightedge or profilograph has been used by several agencies, including the California highway department and the University of Michigan (25, 26) who have truck-mounted versions. The University of Michigan device records a continuous chart profile in each wheel track. Two sets of bogey wheels, 30 feet apart, provide reference points from which vertical displacement is measured by a recording wheel at the midpoint. The cumulative vertical displacement per mile is termed the roughness index. These devices, which check surface roughness of newly constructed pavements, are limited to low operating speeds, and lack of response to wavelengths that are 1/2, 1/4, etc. of the overall wheelbase.

The RRL profilometer has been used in North America by several Canadian agencies, including the Ministry of Transport (30). A schematic of the Ontario unit is shown in Figure 5. It consists of an articulated carriage with four wheel bogies 4 feet wide with a wheel base length of 21 feet. The detector assembly, at the center, consists of a detector wheel mounted centrally on a vertical shaft positioned in vertical guides and trailed by two flanking wheels. These flanking wheels ensure proper tracking of the line of travel. A profile is plotted of the road surface in a natural vertical scale (i.e., the unit is designed so that only 1/16 of the vertical movement of any one wheel is transmitted to the mounting of the detector wheel). The number of bumps of different sizes are measured by means of a classifier (with electrical counters) in intervals of 0.1 inch to 1.5 inches. The roughness value, q, is expressed in inches per mile and is the sum of all downward vertical movements.

The RRL profilometer has excellent repeatability but its applicability is limited by a very low operating speed (i.e., 1 mph). However, it is quite useful for measuring pavement construction not yet opened to traffic and for calibrating other roughness devices such as those of the CRM type subsequently described.

The surface dynamics profilometer (SDP) was originally known as the GMR profilometer, as described by Spangler and Kelley (31, 32). In 1966, K. J. Law, Engineers of Detroit were licensed to manufacture the SDP. Subsequently, much of the additional development and applications of this device have been conducted in Texas (33-36) and in Michigan (37). Figures 6a and 6b schematically show the measurement system and the measurement processing of road profiles with the SDP. Two road-following wheels are mounted on trailing arms beneath the vehicle (see Fig. 6b), one in each wheelpath, and held in contact with the road by a 300-lb spring force. Relative motion between the vehicle and the wheel is measured by a potentiometer. An accelerometer measures the acceleration of the vehicle itself. The signals go into an analog computer in the vehicle. The acceleration signal is then integrated, added to the potentiometer signal, and conditioned (depending on the filter selection) to yield the right and left profiles.

Such profile data in analog form are amenable to power spectral density processing, but other parameters such as slope variance or roughness indexes are difficult to obtain. Consequently, analog-to-digital and digital processing subsystems were also developed (i.e., these other parameters lend themselves to digital processing) to obtain increased flexibility. Detailed descriptions of the overall measuring system and the subsystems are given elsewhere (34, 35, 36).

The SDP has several key advantages or features that distinguish it among roughness measuring devices. These include

1. Determination of actual profiles,
2. Capability of handling large amounts of data by automation
3. Operating speed sufficient to cover reasonable amount of pavement in reasonable time (i.e., 20 mph)
4. Capability of detecting and analyzing longer wavelengths (especially important for airport runways),
5. Excellent repeatability, and
6. Capability of use for calibration of car road meter (CRM) devices.

There are also, however, some disadvantages with the SDP, which include its high capital and operating costs, the need for highly skilled operating personnel, the complexity of the system, the traffic hazard posed, and the computation costs.

The CRM devices have become very popular with highway agencies during the past few years with two major variations. One was originally developed by Max Brokaw (38, 39, 40) and often is called the PCA road meter or the Wisconsin road meter. It has been modified by some people (14, 40, 41, 42). The other device, originally developed by Ivan Mays (27), is now called the Mays ride meter (MRM) and is manufactured by Rainhart Co. (43, 44, 45). A number of evaluation studies

Figure 5. Ontario's RRL type of profilometer (29).

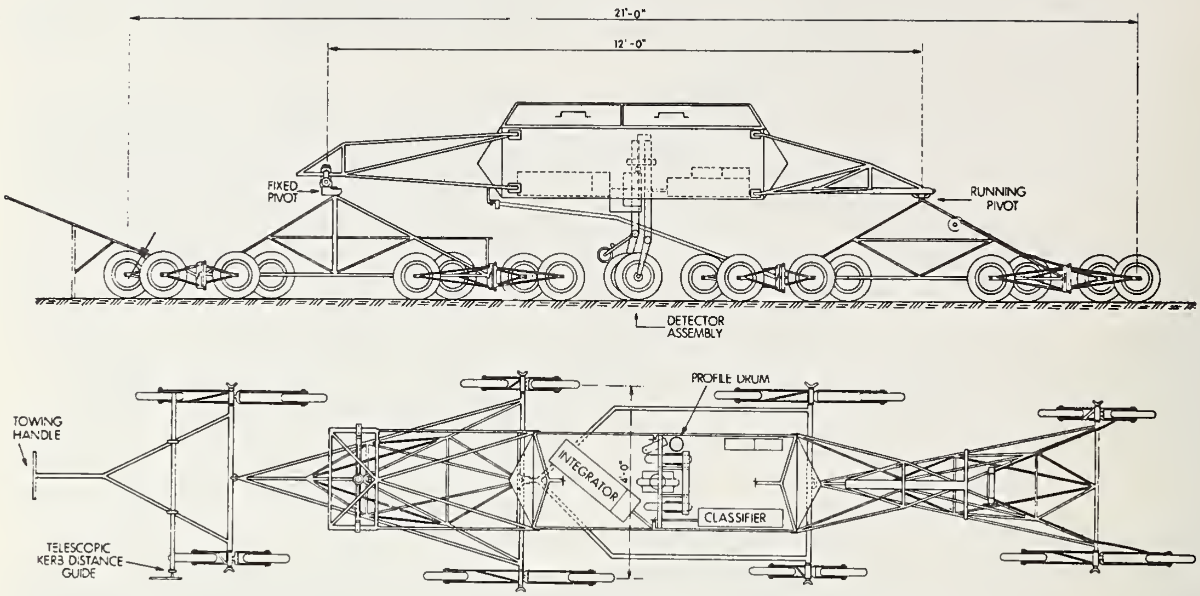
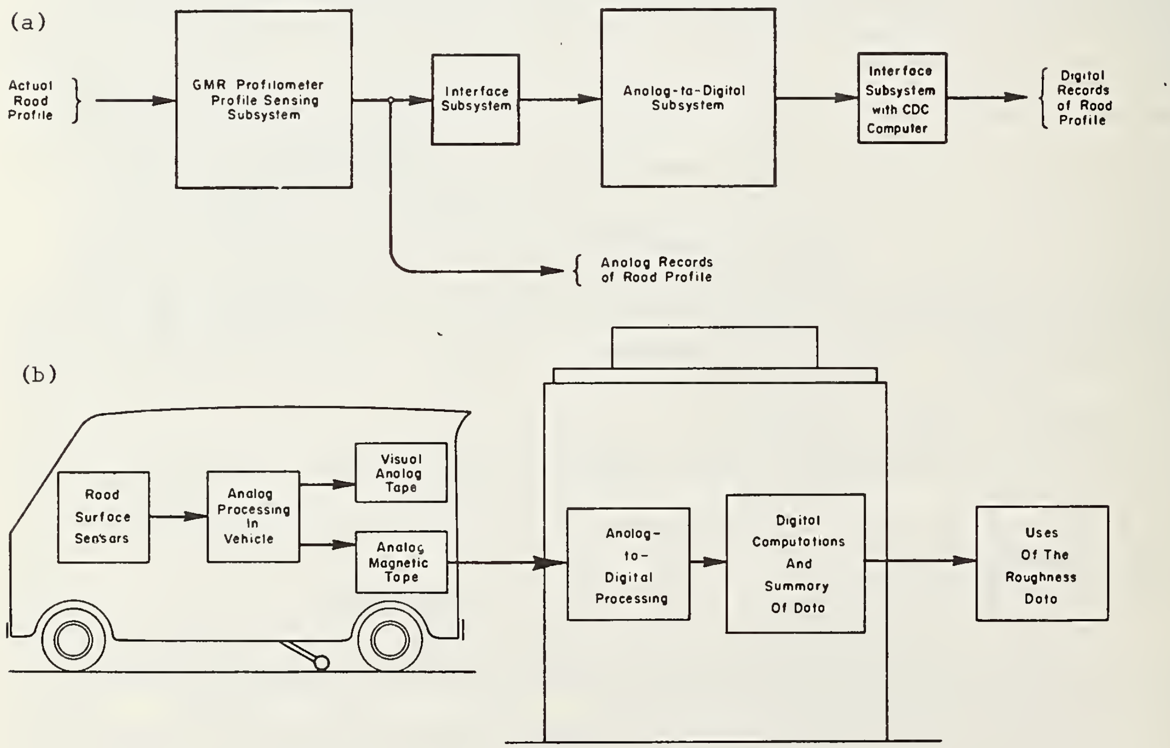


Figure 6. Flow charts of (a) surface dynamics profile measuring system and (b) measurement process by surface dynamics profilometer.



have been conducted on these devices (14, 27, 29, 46-53).

The PCA version of the CRM device, which is schematically shown in Figure 7a, is usually mounted in a late-model automobile or station wagon. It is a simple electromechanical device that measures the number and magnitude of vertical deviations between the body of the automobile and the center of the rear axle. This is accomplished with a flexible cable attached to the rear housing, which passes over a pulley and is restrained by a tension spring, as shown in Figure 7a. A roller type of switchplate, divided into 1/8-inch segments, then records the vertical deviations through a counter system. There are a variety of operating precautions and other features of this device that are reported in more detail in the previously noted references. A very interesting and successful use of the PCA device has been accomplished in conjunction with the photo inventory technique used by the British Columbia highway department (14, 42). Figure 7b is a schematic sketch of the means of incorporating the measurement outputs in a photo frame.

MRM similarly measures the rear axle to body excursions through a photocell sensing system with a 0.1-inch resolution. This system drives a stepping motor for pen and chart drive movements of a 6-inch wide paper tape recorder. The recording pen moves at a rate proportional to the movements of the vehicle body and its differential. Distance traveled is also indicated on the chart by an automatic event marker connected to the speedometer drive. The roughness measurement, which is directly proportional to the total body-differential movement, can be obtained by measuring the amount of chart movement per unit of road length traveled. The paper tape records provide a very useful permanent record of areas that are particularly rough and may require action.

CRM devices have a number of important advantages and these include the following:

1. Relatively low cost, simplicity, and ease of operation,
2. Operating speed (usually 50 mph) at or near normal traffic speed,
3. Capability of acquiring roughness data on a mass inventory basis,
4. Portability of the recording system and continuous strip chart record (with the MRM device), and
5. Reasonably good repeatability.

Disadvantages of the CRM devices include the need for relatively frequent calibration, the need to carefully observe a variety of operating precautions, and the inability to measure profile or long wavelengths. These disadvantages have, apparently, been far overshadowed by the advantages in the opinion of many highway and some airport agencies.

The precise levelling method (i.e., with a surveying rod and level) to determine profile has been the only viable method to obtain actual profile information for airports until the SDP appeared. However, to date the SDP has been used on airport runways in a very limited, experimental basis (54, 55, 56). Although the levelling method is of course very simple and accurate, it is extremely slow and painstaking and requires a long "down time" for the runway section being measured.

Correlating the Outputs of Roughness-Measuring Devices

The correlation of outputs between various roughness-measuring devices is usually done for one or both of the following reasons:

1. Calibration (i.e., using a repeatable device to provide periodic checks for another device that may vary with time or use),
2. Using one device to estimate the output of another.

The following paragraphs briefly outline the types of outputs that are obtained from the devices described in the preceding section and some of the correlations that have been conducted.

The BPR roughometer output (i.e., roughness index in inches/mile) has been correlated reasonably successfully with several other instruments. Examples of such correlations with the RRL device output (i.e., classifier index in inches/mile) and with the PCA type of CRM device output (i.e., sum count, Σ) are shown in Figures 8a and 8b respectively. While these correlations are reasonably good, they may not remain stable because the BPR often has difficulties with repeatability and constancy. Consequently, many highway agencies have gone to the simple, fast, and low-cost CRM devices as an alternative.

The CHLOE profilometer output (i.e., slope variance) has similarly been correlated to other devices. In recent years, this primarily involved the PCA type of CRM device, but has come to include others such as the BPR roughometer (27). Figure 9 is an example of such a CHLOE-PCA correlation from Iowa (57). Similar correlations were obtained in the original development of the PCA device (38) and by others (41, 48). An important point to be noted here is that the PCA type of CRM device is not an instrument for measuring slope variance, as has erroneously been implied in several sources. Rather, it can estimate slope variance, through a correlation such

(a)

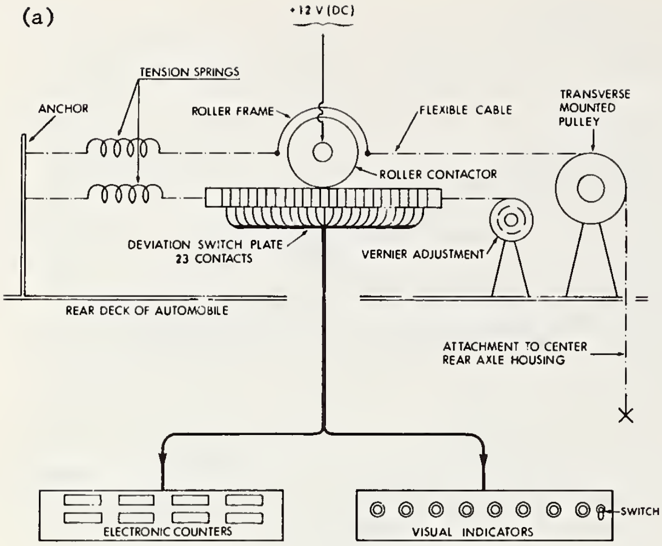


Figure 7. (a) PCA road meter and (b) frame for B.C. photo inventory.

(b)

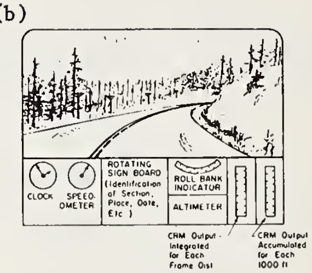
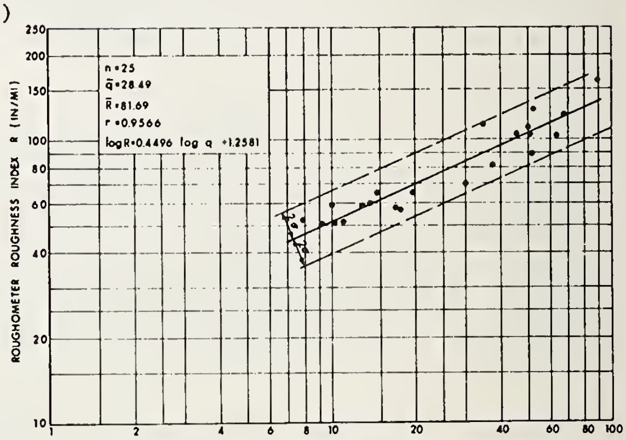


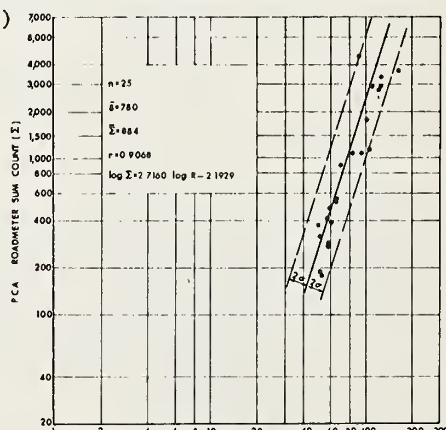
Figure 8. Example correlations of BPR roughometer with (a) RRL profilometer and (b) PCA roadmeter.

(a)



Profilometer Classifier Index Q (in/mi)

(b)



Roughometer Roughness Index R (in/mi)

as shown in Fig. 9.

The RRL profilometer can be a good calibration tool for the CRM device, as indicated by Canadian studies (14, 46, 47, 50, 53). Figure 10 shows example correlations with the RRL profilometer used by the Canadian Ministry of Transport. It reveals that each CRM has its own calibration curve. As might be expected, CRM's correlate quite well with each other (50).

The MRM has shown reasonably good correlations with the BPR roughometer and with the PCA device (27). A typical output of the MRM is shown in Figure 11. Roughness, in inches, is simply scaled off the distance event channel. A significant feature is the roughness signature (i.e., a continuous trace) along the paper tape. This is especially useful for spotting severe bumps for maintenance and other purposes. Extensive correlations of the MRM with the SDP, conducted (45) in terms of estimating serviceability, are discussed in a subsequent section concerned with using roughness measurements to estimate serviceability.

The output of the SDP can be translated into an actual road profile plot. Another means of plotting the output, as used extensively in the Texas work (36), is in terms of a mean power spectral density plot (i.e., mean power in inches^2/cycle/ft. vs. frequency). Spectral analysis, commonly used in such areas as communications engineering, has intrigued pavement researchers for some years because of the capability of considering the entire spectrum of waves in a road profile. However, while it provides a set of spectral values rather than a single statistic such as slope variance, it has been largely of academic interest until recently used in developing equations for estimating serviceability.

Areas of Applicability of Various Roughness-Measuring Devices

It is useful to consider the areas of applicability of the various roughness measurements before discussing their primary end use or purpose of estimating serviceability.

Haas and Hudson (23) have provided a tabular listing of the applicability of roughness measurements. They have suggested that the overall approach should be concerned with purpose of measurement, facility applicability, use of data, and whether the primary interest is estimating serviceability or some other purpose. Table 1 gives an updated version of their listing. There are of course exceptions to this listing, but it should represent common practice for a significant number of North American agencies.

Particular mention might be made of the use of mass inventory roughness measurements for planning and programming pavement rehabilitation and the associated investment requirements. Several highway agencies, notably the British Columbia highway department and the Quebec Ministère des Transports, have placed considerable emphasis on the periodic acquisition of such data and are employing CRM devices for this purpose. Tessier (58) has provided an example of how Quebec plots a continuous record of roughness, along with other pavement evaluation data, as shown in Figure 12. The CRM roughness is in terms of a K-coefficient, as described by Fortin (52). This is an innovative means of using CRM output for direct, linear relation to serviceability without transformations. Computer plots of the road network inventory type shown in Fig. 12 are provided to planning and engineering staff of each administrative district.

Use of Roughness Measurements to Estimate Pavement Serviceability

The major use of objective roughness measurements is to estimate subjective pavement serviceability. Carey and Irick (17) provided the most widely known formula for this purpose in developing the present serviceability index (PSI) equation at the AASHO Road Test. The original form of this equation is as follows:

$$PSI = C + (A_1 R_1 + \dots) + (B_1 D_1 + B_2 D_2 + \dots)$$

where

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

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

R_1 = function of profile roughness [log $(1 = \overline{SV}]$
$\overline{SV}$ = mean slope variance from CHLOE profilometer measurements

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

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

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

D_2 = function of surface deterioration $\sqrt{C+P}$, where $C + P$ = amount of cracking and patching (determined by procedures described in Ref. 28).

Figure 9. Example correlation of CHLOE profilometer with PCA type of CRM device.

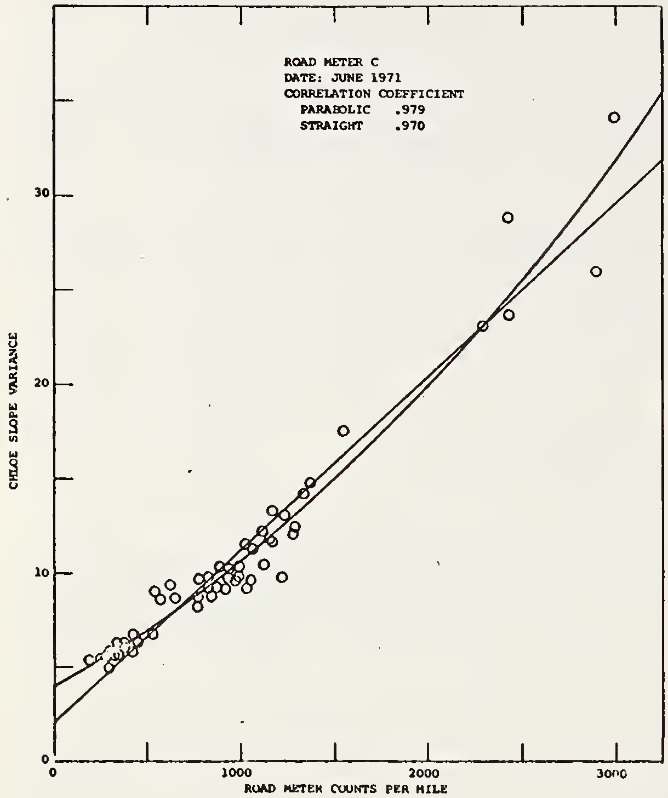


Figure 10. Example correlations of several PCA CRM devices with RRL profilometer.

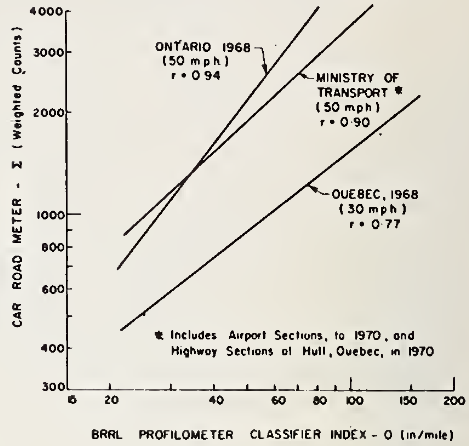


Figure 11. Typical output from Mays ride meter.

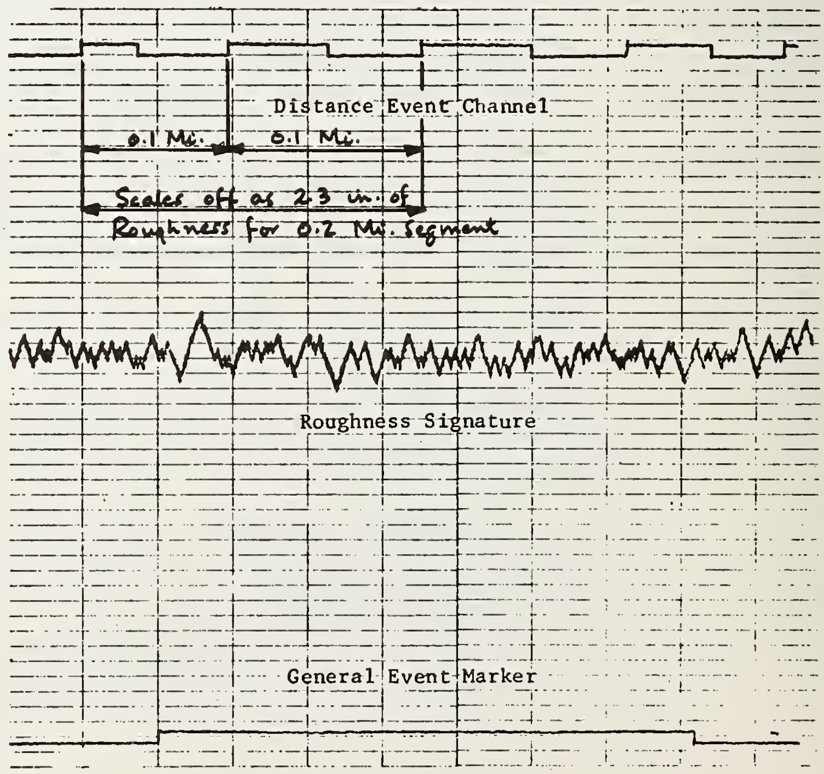


Table 1. Areas of applicability and uses for various types of roughness measurements.

Facility Type	Classes of Measurement, by Purpose: Initial Ride	Periodic Ride	Terminal Ride
1. Expressway or Primary Highway	BPR, SDP, CRM RSE[1] (RRL, CHLOE)[2]	CRM, SDP (RRL, CHLOE)	CRM, SDP (CHLOE RRL)
2. Secondary (Rural) Highway	BPR, CRM, RSE (SDP, RRL, CHLOE)	CRM (SDP, RRL, CHLOE)	CRM (SDP, CHLOE, RRL)
3. County or Local Rural Highway	CRM, BPR, RSE (SDP)	CRM	CRM
4. Runways	SDP, RRL, CRM	CRM, SDP, RRL (LEVEL)	SDP, RRL, LEVEL
Uses of Roughness Measurements, For All Facility Types			
A. Construction Monitoring	Yes[3]	-	-
B. Maintenance Programming	-	Yes	Yes
C. Inventory and Network Programming	-	Yes	Yes
D. Research	Yes	Yes	Yes

[1]See Sec. 3.3.1 for abbreviations of roughness devices

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

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

Figure 12. Portion of Quebec's continuous record of mass inventory data on pavement evaluation.

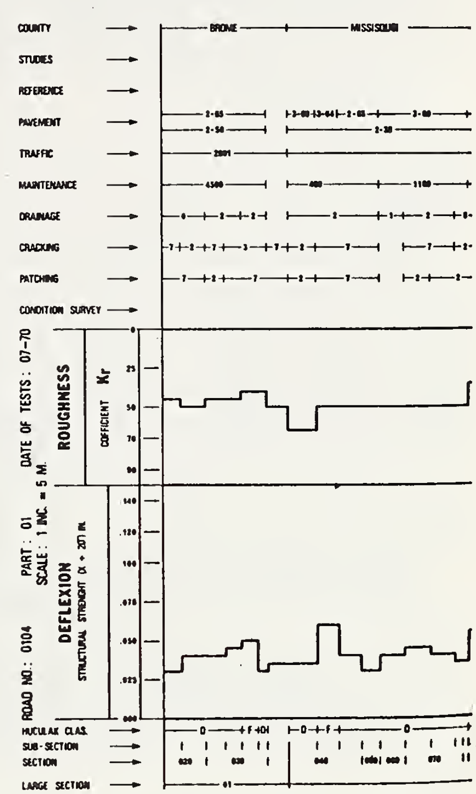
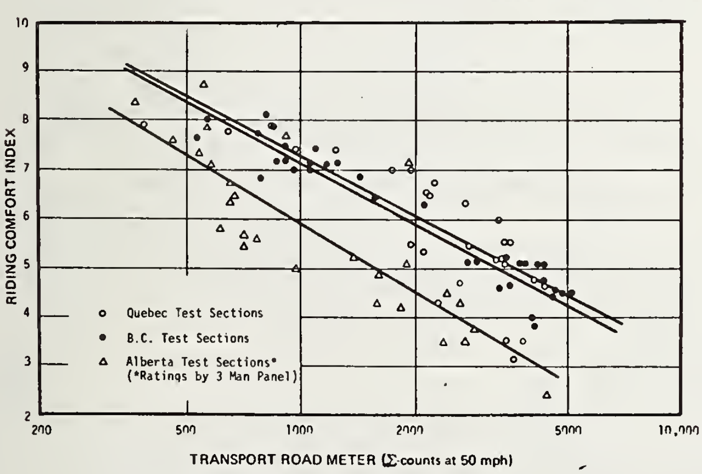


Figure 13. Correlation of riding comfort index with CRM roughness measurements (50).



The PSI equation was developed by multiple regression techniques; i.e., a set of physical measurements was related to the subjective, user evaluations in terms of the mean panel rating values (PSR) described above. While these physical measurements include condition or distress data (i.e., rut depth plus cracking and patching), it is roughness that provides the major correlation variable (i.e., correlation coefficients between PSR and PSI are only increased by about 5 percent by adding in the condition data).

It should be emphasized that whenever PSI is calculated from physical measurement data, this is really only an estimate of PSR, i.e.,

$$PSI = PSR \pm E$$

where E = error term. In other words, contrary to what is frequently implied or stated in the literature, PSI and PSR are not two different ways of obtaining pavement serviceability. PSI represents a means of using objectively obtained data to estimate a subjectively based parameter (17, 23).

The original Canadian evaluation studies previously noted also tried to relate panel ratings to physical measurement data by multiple regression techniques (roughness data were not included). While these efforts were relatively successful in explaining performance variations, the regressions were not significant enough as a predictive design tool for many pavement groups. Consequently, most agencies continued to make direct, periodic subjective ratings until the mid-1960's. At that time, a major program was initiated on relating these subjective RCI values to roughness measurements, primarily using the CRM devices (14). Figure 13 contains example correlations from the Canadian studies. A result of these studies was a set of recommendations relating to correlation and calibration procedures and to operating methods for the CRM's (14, 46, 50, 53).

It should be noted that correlations such as those shown in Fig. 13 can change significantly between regions and with time. Thus the recommendations noted previously have included periodic recalibration experiments.

The PSI equation was developed over 13 years ago and may not be accurate today for some areas in view of the regional and time changes that can occur.

Most efforts by U. S. agencies to correlate CRM output with serviceability have involved several steps. Initially, slope variance of a number of evaluation sections is measured with a CHLOE profilometer. These data are then used to calculate PSI. Next, CRM measurements are taken of the sections, and these are correlated with the calculated PSI. Figure 14 shows such correlations for flexible and rigid sections in Wisconsin (51).

There are two major questions that can be raised concerning approaches that estimate PSI from CRM measurements:

1. The PSI equation itself (which is supposed to estimate PSR) may no longer be valid for the particular area of application.
2. Transformations can compound errors, as demonstrated by Haas and Hudson (23).

As a result, several U. S. agencies have developed their own serviceability equations rather than use the PSI equation (59). The work by Canadian agencies (14) is similar in approach. These efforts are based on the premise that it is necessary to conduct new rating panel sessions at periodic intervals (every 3 or 4 years) and to correlate the results with roughness measurements. The roughness device itself may have to be calibrated at much more frequent intervals.

It should be strongly emphasized that the serviceability-performance concept, as originally advanced (17), has as its principal element the modeling or simulation of subjective user response or opinion. In other words, acceptance of the serviceability-performance concept as the primary output characterization of a pavement does not require acceptance or use of the PSI equation at all. There will undoubtedly continue to be a variety of equations to estimate user opinions, and this is entirely logical in view of the regional differences that exist in user opinions, combined with the changes of these opinions with time. Unfortunately, a great deal of misunderstanding still exists with regard to the foregoing concepts and principles, as exhibited in recent discussions of HRB Task Force A2T59 on relating distress to performance (60). The confusion centers around the fact that performance (i.e., the serviceability-age history) has a precise meaning in the Carey-Irick formulation, while the PSI equation represents only one of the many possible means of estimating serviceability.

Precautions to Be Recognized in Using Subjective Measures

If one accepts the premise that pavements are provided for the user, then user response must be an integral part of analysis and evaluation. Because the methodology for modelling such subjective opinions or ratings has been developed primarily in the field of psychology, engineers are often unaware of its features and its limitations.

The literature on this subject, termed psychophysical scaling, is extensive. Of particular interest to the pavement engineer is an article by Stevens (61), who classified measurements on the basis of the transformations that leave the scale form invariant. Hutchinson (62) and subsequently Haas and Hudson (23) have shown that the considerations presented by Stevens are particularly relevant to the pavement field in terms of the validity of certain statistical manipulations performed on evaluation data. These considerations should be carefully reviewed when experiments to relate subjective user opinions to objective mechanical measurements are devised and when the results are interpreted and applied to design.

There are also several major assumptions involved in acquiring or modelling user opinions, as in the PSR's of the AASHO Road Test or the RCI's of the Canadian studies. These assumptions overlook the following systematic errors that can occur:

1. Leniency error (i.e., a rater's tendency, for various reasons, to rate too high or too low),
2. Halo effect (i.e., a rater's tendency to force a particular attribute rating toward his overall impressions of the object),
3. Central tendency error (i.e., a rater's hesitation to give extreme judgments, thereby tending ratings toward the mean of the rating panel).

A number of guidelines for constructing rating scales and a discussion of the precautions to be used in interpretation have been presented by Hutchinson (67) and by Haas and Hudson (23). They have suggested that careful consideration of these guidelines and precautions can lessen the incompatibilities in pavement evaluation that often exist both within and between agencies.

Project Versus Network Use of Performance Evaluation Data

Pavement performance evaluation data for particular project purposes (i.e., construction monitoring, periodic serviceability estimates, rehabilitation needs) must usually be as precise as possible. To accomplish this, accurate roughness measurements are needed involving the use of devices that are usually slow, expensive, and complex.

Network evaluation data, on the other hand, can be of a less precise nature because they are used for overall needs assessment and rehabilitation investment programming purposes. Consequently, the faster, more efficient, but usually less accurate, roughness-measuring devices, such as the CRM, can be used. This has been the approach of such agencies as the Ministère des Transports in Quebec and the British Columbia highway department (Fig. 7b) in periodically acquiring roughness data on their road networks.

Particular Performance Evaluation Problems and Considerations for Airport Pavements

Airport pavements present several unique problems and considerations in the area of performance evaluation. These include the following:

1. There are two types of users, passengers and flight crew. It is probably sufficient to consider only the flight crew (i.e., in particular the pilot) because their response to roughness is more critical (i.e., primarily with regard to safety) and because they are responsible for passenger safety.
2. The range of aircraft types presents a much wider variation of pavement-vehicle interaction than that occurring on roads.
3. The effects of airport pavement roughness are primarily related to safety and undercarriage damage, as contrasted to roads where they are more related to variation in "quality" of ride provided to the user.
4. In terms of the user, there is no widely accepted measure of performance developed for airport pavements.
5. Long waves are more important on airport runways than on roads because of higher speeds and different wheel and gear configurations.

The effects of "excessive" airport pavement roughness, with respect to porpoising or skipping, slowed acceleration on take-off, cockpit acceleration, difficulties encountered by the flight crew in reading instruments, and structural damage to aircraft, have been well summarized by Alford (63, 64). Aircraft responses, and the possibilities of structural damage, have been considered in a number of publications (65,66).

The main method of measuring airport pavement roughness in the U.S. has been the simple but time-consuming method of rod and level. In Canada, the department of transport has relied on biannual measurements of runway roughness with the RRL. In recent years, CRM devices have largely supplemented RRL measurements, permitting more extensive, mass-inventory measurements to be made. The RRL device is now used mainly for calibration, construction control and special studies.

The surface dynamics profilometer has recently been suggested by Hudson as a useful, potential means for measuring airport pavement roughness (55). It has the capacity to measure roughness in terms of the various wavelengths and amplitudes that are important to pavement-aircraft-pilot interaction, and it is sufficiently fast enough to keep runway downtime to a minimum. Hudson also suggested that some type of serviceability measure, similar in concept to that developed for highways, could be developed for airport pavements. This suggestion was incorporated into an experiment by Steitle (56) who employed a user rating form (with pilots as the users) to evaluate runways at Love Field in Dallas. He also made profile measurements with the SDP and demonstrated that these could be related to the user evaluations.

PAVEMENT CONDITION EVALUATION

Basic Purposes of Pavement Condition Surveys

Most highway and airport agencies conduct periodic condition surveys of their pavements involving mechanistic measurements of distress, such as various types of cracking, ravelling, disintegration, and rut depths. These should not be confused with performance evaluation, which is conducted in terms of the user.

Rather, condition surveys should be directed toward assessing the rehabilitation measures needed to prevent accelerated future distress. They are for use by the "manager" of the pavement and do not simulate user response. They are however related to the user insofar as distress is the cause of both present and future loss of servieability. However, distress measurements as acquired in condition surveys should not be taken to represent user response.

Condition surveys usually involve a fair amount of detail. They generally include not only the type of distress that has occurred but also its degree or magnitude and location.

There are as many methods for conducting condition surveys as there are agencies involved. As stated in a 1968 state-of-the-art report on condition evaluation, "There appears to be no single method of making a condition survey that is used universally. Because of the many uses made of this information, an extremely wide variation exists in the manner in which the surveys are obtained, recorded, analyzed, summarized, and stored" (26). There is a considerable amount of literature available on condition survey methods, including criteria such as that contained in HRB Special Report 30 (67) and the operating manuals of various agencies.

Principal Components of Condition Surveys

Condition surveys measure various types and degrees of distress. There is some degree of commonality among the different methods with respect to the components or factors that are usually measured. These can include the following general factors:

1. Cracking
    a) Type (i.e., longitudinal, transverse, alligator, map, reflection, corner, edge)
    b) Amount or severity
    c) Location
2. Disintegration
    a) Type (i.e., ravelling, stripping, spalling, scaling)
    b) Amount or severity
    c) Location
3. Permanent deformation
    a) Rut depths
    b) Location
4. Distortion
    a) Type (i.e., settlement, heave)
    b) Degree and extent
    c) Location

Some agencies include skid resistance and roughness measurements in what they term condition surveys. Whether or not to include such factors in the definition is an arbitrary choice of each agency. What is important is that they, along with all the other subjective and mechanistic components of pavement evaluation, be considered in assessing rehabilitation needs.

However the combination of mechanistic and subjective evaluations in some rating measure is a mixture of apples and oranges (23). Such a mixture may work reasonably well for a particular agency in being used for rehabilitation decisions but is usually devised by agency staff, based on their experience, and therefore has limited applicability outside the particular agency.

Figure 14. Correlations of PSI with CRM roughness measurements (51).

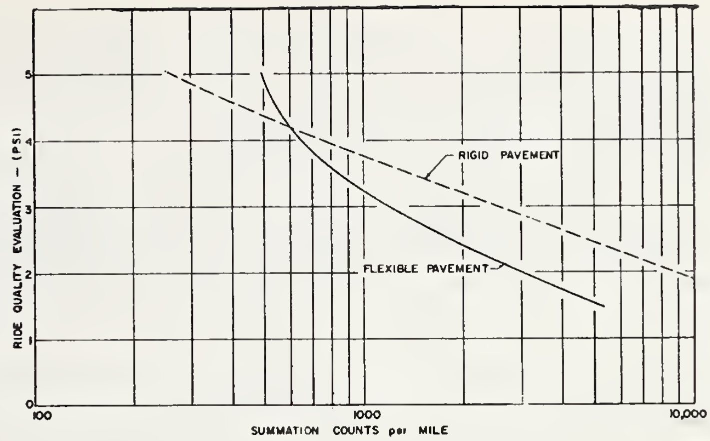
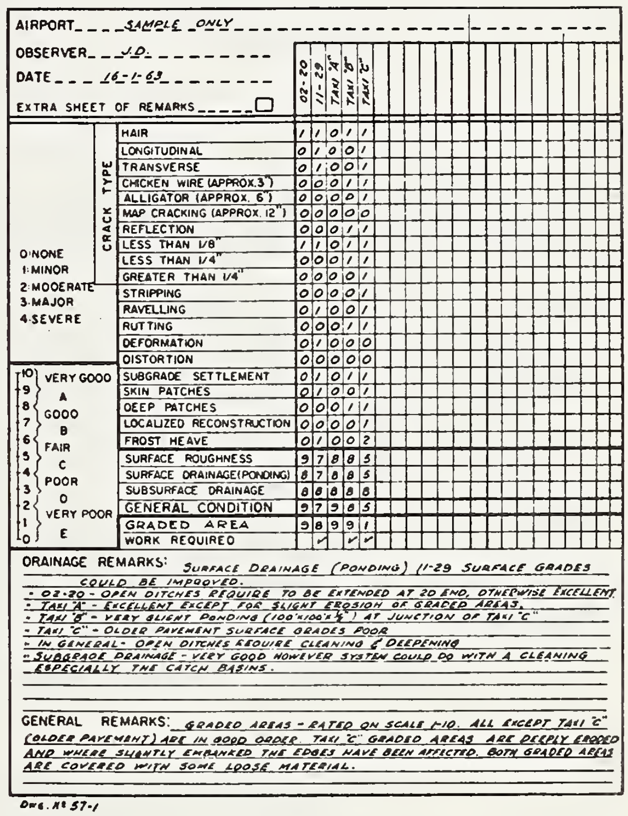


Figure 15. Condition survey reporting form for flexible airfield pavements (71).



AIRPORT ____ SAMPLE ONLY ____

OBSERVER ____ J.D. ____

DATE ____ 16-1-63 ____

EXTRA SHEET OF REMARKS ____ ☐

		02-20	11-29	TAXI "A"	TAXI "B"	TAXI "C"
CRACK TYPE	HAIR	1	1	0	1	1
	LONGITUDINAL	0	1	0	0	1
	TRANSVERSE	0	1	0	0	1
	CHICKEN WIRE (APPROX. 3")	0	0	0	1	1
	ALLIGATOR (APPROX. 6")	0	0	0	0	1
	MAP CRACKING (APPROX. 12")	0	0	0	0	0
	REFLECTION	0	0	0	1	1
	LESS THAN 1/8"	1	1	0	1	1
0: NONE	LESS THAN 1/4"	0	0	0	1	1
1: MINOR	GREATER THAN 1/4"	0	0	0	0	1
2: MODERATE	STRIPPING	0	0	0	0	1
3: MAJOR	RAVELLING	0	1	0	0	1
4: SEVERE	RUTTING	0	0	0	1	1
	DEFORMATION	0	1	0	0	0
	DISTORTION	0	0	0	0	0
10–9 VERY GOOD A	SUBGRADE SETTLEMENT	0	1	0	1	1
	SKIN PATCHES	0	1	0	0	1
8–7 GOOD B	DEEP PATCHES	0	0	0	1	1
	LOCALIZED RECONSTRUCTION	0	0	0	0	1
6–5 FAIR C	FROST HEAVE	0	1	0	0	2
	SURFACE ROUGHNESS	9	7	8	8	5
4–3 POOR D	SURFACE DRAINAGE (PONDING)	8	7	8	8	5
	SUBSURFACE DRAINAGE	8	8	8	8	8
2–1 VERY POOR	GENERAL CONDITION	9	7	9	8	5
0 E	GRADED AREA	9	8	9	9	1
	WORK REQUIRED		✓		✓	✓

DRAINAGE REMARKS: SURFACE DRAINAGE (PONDING) 11-29 SURFACE GRADES COULD BE IMPROVED.
- 02-20 - OPEN DITCHES REQUIRE TO BE EXTENDED AT 20 END, OTHERWISE EXCELLENT.
- TAXI "A" - EXCELLENT EXCEPT FOR SLIGHT EROSION OF GRADED AREAS.
- TAXI "B" - VERY SLIGHT PONDING (100'×100'×¼") AT JUNCTION OF TAXI "C"
- TAXI "C" - OLDER PAVEMENT SURFACE GRADES POOR
- IN GENERAL - OPEN DITCHES REQUIRE CLEANING & DEEPENING
- SUBGRADE DRAINAGE - VERY GOOD HOWEVER SYSTEM COULD DO WITH A CLEANING ESPECIALLY THE CATCH BASINS.

GENERAL REMARKS: GRADED AREAS - RATED ON SCALE 1-10. ALL EXCEPT TAXI "C" (OLDER PAVEMENT) ARE IN GOOD ORDER. TAXI "C" GRADED AREAS ARE DEEPLY ERODED AND WHERE SLIGHTLY EMBANKED THE EDGES HAVE BEEN AFFECTED. BOTH GRADED AREAS ARE COVERED WITH SOME LOOSE MATERIAL.

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Figure 16. British Road Research Laboratory procedure for individual and overall surface condition reassessment (72).

1	2	3	4	5	6	7	8		9
Section No.	Disintegrating ⟷ Excess Binder 1, 2, 3, 4, 5, 6, 7	Deformation None ⟶ Very Severe 0, 1, 2, 3	Traffic Laning None ⟶ Very Severe 0, 1, 2, 3	General Variability None ⟶ Very Severe 0, 1, 2, 3	Texture R→M→S	Overall Condition VG⟶B	Average assessment as later agreed: Overall Condition	Texture	Remarks
104	4	0	0	0	M	VG	G	MR	
105	4	0	0	0	M	VG	G	M	
106	5	0	0	0	M	G	G	M	
107	5	0	0	1	M	G	G	M	
108	6	0	1	1	S	FG	FG+ε	MS	
109	6	0	2	1	S	FG	FG÷ε	MS	
110	5	0	1	1	M	G	G	M	

The format of the necessary reporting forms for actually conducting condition surveys and processing the data varies among agencies. Examples can be found in their operating manuals. Suggested forms for both flexible and rigid pavements may be found in the references (67).

Examples of Condition Survey Procedures

A brief consideration of two representative examples of condition survey procedures is useful to amplify the general discussion of the preceding sections. One of the better known methods is that used in Washington (68, 69). It consists of, in part, what is termed a "defect rating," which is determined by a condition survey conducted by a two-man crew. There are established categories of defects on the rating forms, and guidelines are used to judge their seriousness. Numerical weighting values are assigned, in an approach similar to that used by the Saskatchewan highway department (70). These are then used to calculate the defect rating, G_D, as follows:

$$G_D = 100 - \Sigma \text{ (defect values for pavement distress)}$$

The Washington method also uses a ride rating, G_R, determined by the two-man crew. A range of 0 to 10 is used with 0 indicating a perfect ride and 10 a virtually impassable road. The ride rating is then calculated as follows:

$$G_R = 100 - (10 \times \text{ride score})$$

Both the defect rating and the ride rating in the Washington method are then combined to give a final rating, R_R, according to the formula:

$$R_R = \sqrt{G_R \times G_D}$$

Condition surveys for airport pavements are highly important parts of the total evaluation needs of these surfaces. One such method is used by the Canadian department of transport (71). Their reporting form for flexible pavements (Figure 15) illustrates the various factors that are measured and the assigned weights. Photographic records of major distress conditions are also kept and provide a useful supplement.

An example of a condition survey method used in Britain and developed by the Road Research Laboratory is shown in Figure 16 (72). The overall condition is reported in descriptive terms rather than a numerical basis.

Photo inventory methods, such as that used by British Columbia (see Fig. 7b), can be used to provide a considerable amount of condition information. However, they are not, in their present stage of development, satisfactory for detailed evaluation of such distress conditions as hairline cracks, ruts, etc.

The condition survey methods discussed previously and the many others in existence, are only applicable to particular regions and jurisdictions. Nevertheless, examining them can provide useful comparisons for any individual method (largely with respect to the factors measured and their importance) or useful guidelines to an agency wishing to modify its method or develop a new one.

PAVEMENT SAFETY EVALUATION

Major Safety Components

The evaluation of pavements for safety usually considers only slipperiness (in terms of skid resistance). There are, however, several components of safety evaluation including:

1. Skid resistance,
2. Ruts (as they relate to accumulation of water and the dangers of hydroplaning or ice accumulation),
3. Light reflectivity of the pavement surface,
4. Lane demarcation, and
5. Debris or foreign objects (especially relating to airport pavements).

There are situations where any one of these factors can constitute a significant safety hazard; however, pavement slipperiness seems to be the most common and widespread. Consequently, it has been selected for particular consideration in the following discussion.

Skid Resistance Evaluation

The phenomenon of skidding involves a very complex interrelationship among pavement, vehicle, (mainly the tire), and driving factors. Nevertheless, a great deal of progress has been made during the past 2 or 3 decades in understanding the phenomenon and in developing measurement techniques and evaluation procedures. As a result, the area of skid resistance evaluation, particularly with respect to its purpose, is perhaps better understood than that of performance evaluation.

Thus, the following discussion concentrates primarily on methods of measurement and data uses rather than on the factors that cause skidding and the methods for either providing better skid resistance or remedying unsafe conditions.

Basic Concepts of Skid Resistance and Uses of Measured Values

The coefficient of friction, μ, as used in the solid mechanics field, is calculated by dividing the frictional resistance to motion in the plane of the interface, F, by the load acting perpendicular to the interface L. It is dependent on the contact area and therefore not a suitable representation of the actual pairing that occurs between tire and pavement surface. In other words, frictional resistance of pavements is not just a pavement property.

There are in fact various conditions additional to the pavement itself that influence the rolling, slipping, or skidding of a tire, particularly when water is present. Thus, the term preferred in the pavement field is "friction factor", f, which is calculated as

$$f = F/L$$

Because it is not correct to ascribe a particular f to a pavement without specifying the tire, speed, temperature, water film thickness, and other conditions that influence it, standards have been developed for measurement. The best known standard is ASTM Method E#274 and measurements made in accordance with it are termed skid numbers (SN), which are calculated as follows:

$$SN = 100f = 100\ F/L$$

Skidding accidents occur not only by direct forward sliding (i.e., as in an emergency stop) where all wheels are locked but also by jackknifing (i.e., where only one or one set of wheels locks) and by breaking away or sliding off curves. Most such accidents occur under wet or icy conditions. Thus, skid resistance measurements are conducted on wetted pavements.

Skid resistance measurements that are taken by some standardized procedure, such as ASTM Method E274, cannot indicate the precise value available to a particular vehicle and driver. This varies with the type of tire, the amount of tire wear, the maneuver (i.e., acceleration, deceleration, lane change), in addition to the drainage and texture characteristics of the pavement surface. Nevertheless, an SN value obtained by the standardized method can give a reasonably qualitative representation for a significant portion of the traffic.

The large amount of literature available in the area of skid resistance has recently been well summarized (73), considering not only the various techniques of measurement but also skid resistance requirements and various design or remedial actions that may be taken. As well, it has been pointed out that skid resistance data can be used for the following pavement management purposes:

1. Identifying areas of excessive slipperiness,
2. Planning maintenance, and
3. Evaluating various types of materials and new construction practices.

Skid resistance measurements might additionally evaluate such factors as the effects of studded tires.

Methods of Measuring Skid Resistance

Many skid resistance measurement methods involve measuring the force required to drag a non-rotating tire over a wet pavement. There are also other methods, however, that are equally valid.

Locked-wheel trailer methods are represented by the ASTM Method E274. A bias-ply 7.5 X 14 tire is specified and the towing speed is 40 mph. After the test wheel is locked and has been sliding for a suitable distance, the force is measured and the skid number is calculated for that part of the pavement. Many agencies in North America have skid trailers.

Automobile methods are seemingly the most logical, but they are also dangerous when locking of all four wheels is sued. By using front-wheel or diagonal wheel braking only, the instability is reduced (74). A major disadvantage of automobile methods is the variability in results due to vehicle effects (i.e., differences in suspension, weight, tires, condition, load distribution, air drag). As well, a separate water supply for wetting the pavement can significantly increase the costs of such tests.

Portable field testing methods are perhaps best represented by the well-known pendulum type of device developed by the British Road Research Laboratory (75) and covered by ASTM Method E 303. It involves dropping a spring-loaded rubber shoe attached to the pendulum. The results are reported as British pendulum numbers (BPN). Major advantages of this device include its applicability to laboratory testing, its simplicity, low cost, and easy transportability. However, its use in the field requires the diversion or stopping of traffic. As well, because the shoe contacts the pavement at a relatively low speed, the results do not correlate well with locked-wheel trailer test results conduced at 40 mph.

There are several devices that measure skid resistance in the "slip mode." This relates to the phenomenon that occurs when a wheel is gradually braked, with increasing friction factor, to a point of "critical slip" beyond which the wheel locks and the friction factor drops. The friction factor is therefore higher, and this has practical significance to the development of automatic brake control systems. NCHRP Synthesis Report 14 illustrates that the ratio of f_{max} (i.e., at critical slip) to f_{lock} varies with surface texture, friction force, and temperature. It also makes reference to the "Skidometer" developed by the Swedish Road Research Institute, the runway friction tester with adjustable slip used by the FAA and developed by Meyer and co-workers at Pennsylvania State University, and a recent elaborate device developed in Switzerland.

A variation of the slip mode testing approach involves the direct use of a decelerometer in a test vehicle. One such device that is extensively used for airports in Canada is the James brake decelerometer (76, 77, 78). Most of Canada's skid problems are due to surface contaminants (i.e., ice, snow, slush, water), and this device offers a practical, rapid means for evaluation. The parameter measured is deceleration in feet/second^2, which can vary from 0 to 32. It is reported as a James brake index (JBI). Charts have been developed to translate the results into required landing distance, for various headwind and crosswind speeds. Some NASA studies, as reported by Yager (79), have also considered the effects of contaminants, the applicability of various skid-measuring devices, and the landing distances required for various conditions.

There are also several devices that measure skid resistance in the "yaw mode," where the wheels are turned at some angle to the direction of motion. The side or cornering force is measured and it (as well as the side friction factor) peaks at some yaw angle. Thus, the measurement of pavement friction in this manner presents similar problems as those in slip mode measurement. The critical yaw angle is subject to the same variations as critical slip. Consequently, it is desirable to use an angle that is relatively insensitive to variations in surface characteristics and operating conditions.

A fairly simple version of a yaw mode device is the mu-meter, currently being used and evaluated by several agencies such as the Utah highway department. It was originally developed and tested in Britain and uses two yawed wheels, at 7 1/2 deg., which provides balance.

Another, more sophisticated device is the sideways-force coefficient routine investigation machine (SCRIM) developed by the Transport and Road Research Laboratory in Britain (80). It uses a yaw angle of 20 deg (rather than the 15 deg noted in NCHRP Synthesis Report 14) on the test wheel, which can be lifted clear of the road when not in use. The friction factor measurements are reported as sideways-force coefficient (SFC) numbers where

$$SFC = 100f$$

which is numerically the same as SN.

The major advantages of a device such as SCRIM include the continuous record of skid resistance and high operating speeds. These are particularly important for airport pavements. The major disadvantage of the SCRIM device is the relatively high initial cost. Figure 17 contains a sample output printed on paper strip (80). The first 10 digits are used for identity, and the eleventh is used for events. All manual control of data is by push button. The twelfth and thirteenth digits show the average SFC number for the section lengths chosen, which in this case are 10 m long as shown in the last row of digits. Five- and 20-m section lengths can also be used. Speed varies from 55 to 56 km/h in the example.

A unique approach to indirectly estimating skid resistance has been developed by Schonfeld (81). It involves the use of color stereophotographs to analyze the "texture elements" of the surface in terms of a "texture code number." This has been correlated with skid tests using an ASTM skid trailer, as shown in Figure 18. The correlation seems reasonably good in that the standard deviation of SN ranges from about ± 3 for 30 mph to ± 2.2 at 60 mph. While the method is relatively slow and depends largely on operator skill, it is a very simple and low-cost tool. It is particularly advantageous in evaluating the skid resistance of sections that cannot readily be tested by other means (i.e., at sharp curves or at stop-sign locations) or for use in situations where a skid-testing device is unavailable or impractical to acquire.

Correlations Between Skid Measurements

Correlations between the outputs of various skid testing devices can, as for roughness evaluation, be used for the following purposes:

1. Calibration (i.e., using one repeatable and stable device for periodically checking the output of another), and
2. Using one device to estimate the output of another.

Unfortunately, correlations between various skid measurement devices are not usually very good. This is largely because each device measures a different aspect of the frictional interaction between vehicle and pavement.

While some correlations may be reasonably good, over a limited range of conditions, they "should be considered fortuitous, rather than as a fulfillment of a justified expectation" (73).

Change of Skid Resistance With Time, Traffic, and Climate

Skid resistance evaluation, especially for the purpose of assessing future rehabilitation needs, should consider changes on a time and traffic as well as a climatic effect basis. The latter can involve both short and longer periods of time (i.e., rainfall or icing of a short duration, versus seasonal changes in climate).

These considerations of time/traffic/climate based changes in skid resistance require periodic measurements, preferably on a mass inventory basis for investment programming purposes. Various changes in the nature of the pavement surface should be recognized as possible contributors to such skid resistance changes and they include the following:

1. Porosity,
2. Wear (due to studded tires),
3. Polishing of surface aggregate,
4. Rutting (due to compaction, lateral distortion, or studded tire wear),
5. Bleeding, and
6. Contamination (i.e., rubber, oil, water).

On a short-term basis, skid resistance changes can occur rapidly, usually because of rainfall (Figure 19). On a seasonal basis, skid resistance may fluctuate as shown in Figure 20. On a longer basis of several years or several million vehicle passes, most pavements show a continual decrease of skid resistance, as illustrated in Figure 21.

The method of predicting skid resistance changes with time and/or traffic can be done in either of two basic ways:

1. Extrapolate an existing set of data, acquired over some period of time, into the future, or
2. Conduct laboratory experiments either at the initial design stage or prior to designing rehabilitation measures i.e., polishing characteristics of the aggregates, texture, shape, size, laboratory track tests on simulated mixtures (this approach essentially provides only a qualitative link to estimates of actual skid resistance).

PAVEMENT EVALUATION WITH RESPECT TO USER COSTS AND BENEFITS

Rehabilitation Based on User Costs or Benefits

It has been suggested that rehabilitation decisions can be based on one or more of the following: need for greater structural capacity, excessive distress, unacceptable maintenance costs, and excessive user costs. In actual practice, such decisions are almost always based on structural, serviceability, distress, safety, or maintenance cost considerations. However, excessive user costs arising from certain surface characteristics can be an equally valid criterion for rehabilitation needs. Unfortunately, this important area has received scant attention in the pavement field.

The practice has been to consider only capital, maintenance, and engineering or administrative costs, with the implicit assumption that user costs do not vary with level of serviceability, surface distress, extent and time of rehabilitation, and so on. In other words, all initial design strategies, and any subsequent rehabilitation strategies or measures, are assumed to result in equal user costs. However, this approach is inadequate because user costs can vary

Figure 17. Portion of paper strip output from SCRIM device (80).

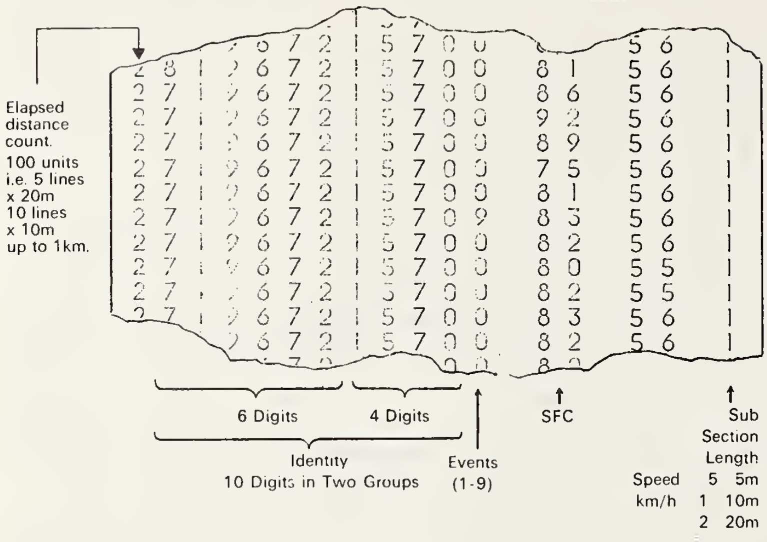


Figure 18. Correlations of skid numbers measured by ASTM trailer and estimated from stereo-photograph analysis (81).

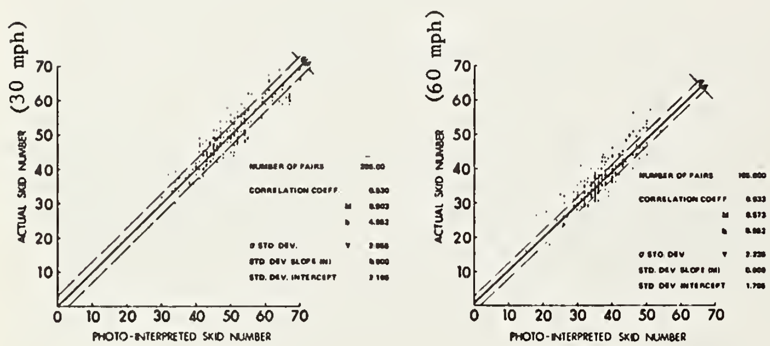


Figure 19. Change in skid resistance because of rain (73).

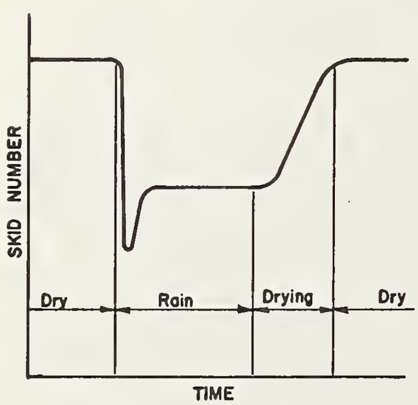
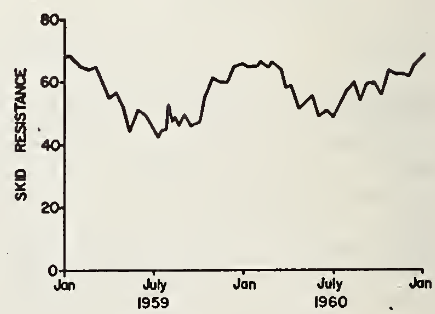


Figure 20. Seasonal change in skid resistance (73).



significantly with several pavement factors (82). In turn, because cost reductions can be considered as savings or benefits in highway economy analyses (83), the analysis for economy of pavements should include user costs and benefits (84).

User costs associated with pavement surface characteristics can be subdivided into the following components:

1. Vehicle operating costs,
2. Travel time costs,
3. Accident costs, and
4. Discomfort costs.

In addition, there are certain user costs associated with rehabilitation operations themselves, primarily due to travel time delays, that must be considered in the total evaluation.

Variation of User Costs With Pavement Surface Characteristics

It has been previously suggested that user costs can be affected by pavement surface characteristics such as level of serviceability, slipperiness, appearance, distress. The first two of these are probably more important than the others for most situations. Considering only level of serviceability, for example, Figure 22 is a schematic representation of the effects of three performance profiles (as shown by pavement strategies i, j, and k) on user costs. The user costs are subdivided into (a) travel time costs and (b) an aggregation of vehicle operation, accident, and discomfort costs. Travel time costs are separately identified to illustrate two points:

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

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

1. As pavement serviceability approaches a terminal level, user costs increase sharply;
2. Strategies that do not call for resurfacing until a lower limit of terminal or minimum acceptable serviceability is reached (i.e., strategy k of Fig. 22) will result in higher user costs.

The accurate quantification of user costs as a function of such factors as pavement serviceability requires considerable research effort. In fact, McFarland (82) has been the first to actually provide some preliminary estimates. He considered the variation of serviceability in terms of speed, type of area (rural or urban), and type of facility (two-lane, four-lane undivided, or four or more lanes divided), as shwon in Figure 23. Using these relationships plus other available information on vehicle operating costs, travel time costs, accident costs, and discomfort costs, McFarland developed tables and curves of user costs.

PAVEMENT SURFACE EVALUATION WITH RESPECT TO NOISE EFFECTS

Highway noise effects have been extensively studies by a number of agencies. These efforts have, however, almost invariably concentrated on outside effect, i.e., related to the nonuser. As well, the studies have involved such factors as vehicle speed and type, geometric configuration of the facility, wind, etc. and they have examined various measures for alleviating excessive noise levels as experienced by nonusers. In fact, many agencies have legislated limits on outside vehicle sound levels.

Within-vehicle noise and the effects of the pavement characteristics themselves, as they might relate to pavement rehabilitation, have received comparatively far less attention. It is likely that most agencies consider this a very minor aspect of pavement evaluation. Nevertheless, there is some limited information available on within-vehicle noise and the effects of the pavement.

Noise level tests conducted by Popular Science in 1971 on five small cars examined the effects of a "smooth road" and a "rough road" at 30 and 60 mph (85). The results, in decibels, were as follows:

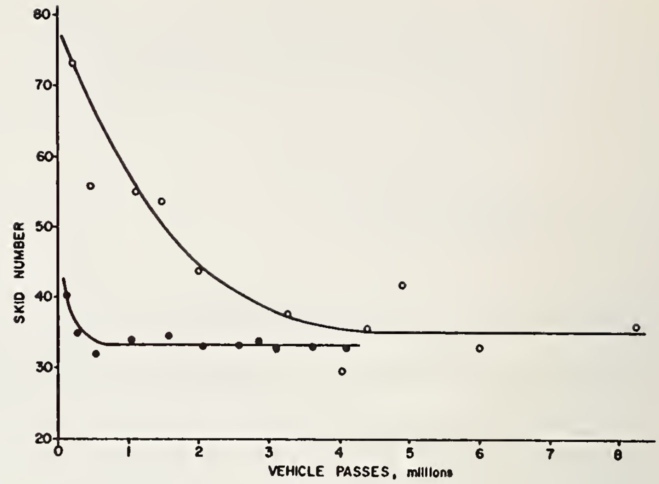


Figure 21. Skid resistance loss of two pavement sections with traffic applications (73).

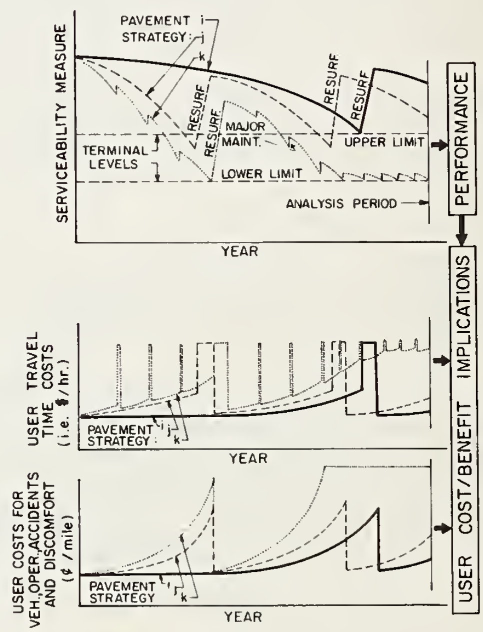


Figure 22. Effects of various performance profiles on user costs.

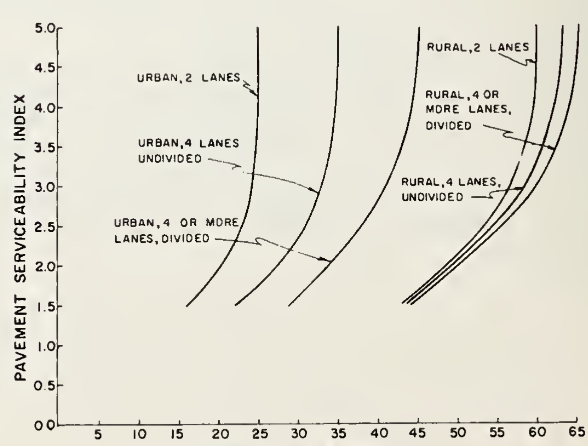


Figure 23. Average vehicle speed related to pavement serviceability index by road type.

Car Type	Smooth Road 30 mph	Smooth Road 60 mph	Rough Road 30 mph
Fiat	66	76	81
Subaru	73	79	78
VW	70	79	79
Vega	68	83	82
Pinto	67	78	84

It appears that the rough road effect can be relatively severe in increasing within-vehicle noise for all the cars tested. The range of increase, 5 db for the Subaru and 17 db for the Pinto, are all highly noticeable (i.e., a 1-db difference is perceptible to a driver, while a 5-db difference is highly noticeable).

The effects of pavement grooving on within-vehicle noise have been studied by Maynard and Lane (86). For nongrooved roads, tests at 30 and 60 mph on several widely different types of surfaces revealed a change of only 6 db. On the grooved surfaces, sound levels did not change appreciably except at 40 mph for vehicles fitted with raidal ply tires where an objectionable "whine" occurred. They have used the results to recommend changes in grooving patterns.

SUMMARY AND CONCLUSIONS

Rehabilitation measures for highway or airfield pavements are usually based on one or more of the following considerations:

1. Inadequate structural capacity,
2. Unacceptable level of serviceability,
3. Unacceptable level of distress,
4. Unacceptable level of safety,
5. Unacceptable user costs, and
5. Unacceptable routine maintenance costs.

This paper has presented a state-of-the-art review of the evaluation procedures used to assess the serviceability, distress, safety, and user cost considerations.

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A REVIEW OF
STRUCTURAL EVALUATION AND OVERLAY DESIGN FOR
HIGHWAY PAVEMENTS

Richard A. McComb and John J. Labra

The structural evaluation of a pavement and the subsequent design of an overlay to correct deficiencies or to provide for future traffic are an integral part of the overall pavement rehabilitation framework. The determination of the overlay thickness is a very important aspect of pavement rehabilitation, but many other items are significant, and unless adequate attention is given to some of these, the best of designs may be a failure.

Various overlay design procedures are available and are being used by a number of agencies. These are well-documented procedures and can be put to use by others. Some are less structured and would require some adaptation based on local experience to be adopted for for general use.

The best knowledge and information have generally been used in the development of each procedure and usually have included a look at the existing pavement structural design method, results of theoretical work in the area, and the published works of others. These methods have been combined into usable overlay design method and are continually being upgraded and improved and, in some instances, the published version may be obsolete.

Most agencies responsible for the maintenance and upgrading of pavements have procedures (formal and informal) for evaluating pavements and determining overlay thickness. An attempt has been made to describe the procedure, the data necessary to use it, its application and limitations, development of the procedure, and assumptions made. Because most procedures have the structural evaluation of the existing pavement built into the method, this is also described. Where applicable, remaining life analysis is also included.

PORTLAND CEMENT CONCRETE OVERLAYS

Several overlay design procedures have been published for concrete resurfacing of pavements. The Portland Cement Association, American Concrete Institute, and the Corps of Engineers (1, 2, 3) have detailed methods. They are all similar modifications of existing design procedures and determine the thickness of the PCC overlay required if a single PCC slab is used. Determination of this thickness is arrived at by using normal PCC pavement design procedures and is independent of the overlay design. An agency's standard design procedure, such as AASHO or PCA, can be used. The formulas used in the overlay procedure have been developed by the Corps of Engineers for airfield pavements and have been used for highway pavements. Details of their development will not be discussed here. In general PCC overlays can be classified as modifications of structural design procedures. Concrete overlays may be grouped into three types depending on the bond, condition between pavement and the overlay.

Bonded Overlays

Special preparation of the existing surface is needed to ensure complete bond with the overlay. The surface must be completely clean and etched, and a bonding agent used. When complete bond is ensured, the overlay and base slab act as a monolithic slab. The overlay thickness, Tr, is given by

$$Tr = T - To$$

where T is the thickness required and To is the existing thickness. Because of the complete bond between layers, underlying cracks are expected to reflect through the overlay. Matching of joints as to type and location is also required. A minimum thickness of 1 inch is mentioned in the ACI procedure; however, bonded overlays should only be put over structurally sound pavements. Limited structural defects need to be repaired prior to overlay.

Unbonded Overlay

At the other extreme are the unbonded overlays where separation courses are placed over the existing pavement to prevent bond. The overlay thickness for unbonded overlays is given by

$$Tr = \sqrt{T^2 - To^2}$$

This is theoretically correct when the old pavement and the overlay are the same stiffness and act as two independent beams deflecting equally under load with no bond between them.

Because existing concrete pavements vary considerably in their structural condition, it was found necessary to introduce a coefficient, C. The value of C applies to the structural conditions only and should not be influenced by surface defects such as surface cracks, sealing, spalling, and shrinkage cracks. The practice has been to use the following values of C:

1. C = 1.0 when the existing pavement is in good overall structural condition,
2. C = 0.75 when the existing pavement has initial joint and corner cracks due to loading but no progressive structural distress or recent cracking, and
3. C = 0.35 when the existing pavement is badly cracked or shattered structurally.

The overlay thickness for unbonded overlays, using the coefficient C, then becomes

$$T_r = \sqrt{T^2 - CTo^2}$$

Because of the separation course, reflection cracking is not normally expected, and it is not necessary to match joint locations in the overlay. A minimum thickness of 6 inches is recommended for unbonded overlays. This overlay is the only one that can be placed over existing pavements with severe structural defects.

Partially Bonded Overlays

When no special steps are taken to prevent bond between the overlay and the existing pavement, a partial bond may develop, and the two slabs will act as an integral unit. Traffic tests on overlay pavements placed directly on existing slabs have shown that the unbonded thickness formula produces a thinner overlay. The Corps of Engineers developed the following formula for designing overlays placed directly on the existing pavement:

$$Tr = 1.4 \sqrt{T^{1.4} - To^{1.4}}$$

The use of this formula results in overlays thinner than those obtained by using the unbonded formula. Friction between the slabs causes the system to have somewhat greater capacity. As in the case with the unbonded overlays, the structural condition of the existing pavements varies, and the overlay thickness for partially bonded overlays, using the same coefficient, C, then becomes

$$Tr = 1.4 \sqrt{T^{1.4} - CTo^{1.4}}$$

where C is a measure of the structural condition of the existing pavement.

Because the overlay is partially bonded and the layers perform as a unit, reflection cracking will usually occur, and the jointing pattern in the overlay must match the existing pavement. A minimum thickness of 5 inches is recommended and, as is the case with bonded overlays, partially bonded overlays should only be put over structurally sound pavements and limited structural defects must be repaired prior to overlay. Fig. 1 summarizes the procedures for the different bond conditions.

FIBROUS CONCRETE

Considerable attention is being focused on the use of fibrous concrete for highway overlays. A proposed design criterion for fibrous concrete overlays over existing rigid pavement has been developed for the Corps of Engineers (14). This design follows the general procedures for conventional rigid overlays. Only differences from the conventional method will be discussed.

For bonded overlays with fibrous concrete, a shift in the neutral axis will occur because of different material properties of the fibrous overlay. If identical materials are used for the base pavement and overlay, the strain distribution would be linear and the neutral axis would be at the middepth of the combined thickness. The full potential of fibrous concrete cannot be realized since the material is used as an overlay in the compressive zone of the cross section. The recommended equation to determine the thickness of a bonded fibrous concrete overlay is, therefore

$$T_f = 0.9Tr = 0.9\ (T-To)$$

where T_f is the thickness of fibrous concrete overlay. For unbonded and partially bonded overlays with fibrous concrete, the thickness determined by the conventional procedure should be reduced by one-half. The overlay thickness for unbonded fibrous overlays then becomes

$$T_f = 0.5Tr = 0.5\ \sqrt{T^2 - CTo^2}$$

and the overlay thickness for partially bonded fibrous concrete overlays is

$$T_f = 0.5Tr = 0.5^{1.4}\ \sqrt{T^{1.4} - CTo^{1.4}}$$

The one-half reduction in required thickness is due not solely to differences in flexural strength, but also to the result of post-crack behavior of fibrous concrete.

Fibrous concrete overlays of existing flexible pavement should be treated as slabs on grade and design procedures developed by Rice (4) should be used.

FLEXIBLE OVER RIGID

A Corps of Engineers procedure for flexible overlays over rigid pavements is also available (15). It is based on an empirical formula that assigns a structural equivalency to relate the thickness of asphaltic concrete and PCC.

$$t = 2.5\ (Fh_d - Ch)$$

where

t = thickness of the overlay (inches),
F = a factor related to subgrade modulus,
h_d = the design thickness determined from the regular PCC pavement design procedures (inches),
h = existing thickness (inches), and
C = a factor related to condition of existing slabs.

The F value is a function of the modulus of subgrade reaction and the type of loading expected. A critical step in this procedure is the selection of the proper F value inasmuch as it decreases as the support value increases. The manual gives guidance for selecting the F value for general conditions of traffic loading, as shown in Fig. 2.

CONTINUOUSLY REINFORCED CONCRETE OVERLAYS

Continuously reinforced concrete pavement (CRCP) is defined as a concrete pavement in which the longitudinal reinforcing steel is continuous for its length and no transverse joints other than construction joints are installed. The principles of design for CRCP according to McCullough (6) are (a) to provide sufficient steel to ensure that the cracks in the concrete are small enough to prevent passage of surface water downward into the underlying material and to provide adequate aggregate interlock for load transfer across the crack and (b) to keep the flexural stresses in the pavement below a predetermined allowable level. Procedures that determine the thickness and reinforcement requirements are available for design of new CRC pavements.

The design concepts for CRCP overlays for existing concrete pavement are essentially the same as for the design of a new CRCP. The principle difference in the method presented by McCullough is in the design charts for pavement thickness. These charts take into account flexural strength of the concrete, wheel load, subgrade support, and thickness of the existing pavement.

Prior to placing the CRCP overlay, it is recommended that the existing concrete pavement be overlaid with a 1/2- to 3-inch layer of bituminous mix. The bituminous overlay serves as a stress relieving layer placed to ensure that the volume change movements of the original slab will not induce stresses in the new CRCP overlay. When the existing pavement is structurally sound and the joint spacing is 15 feet or less, a bituminous overlay is not required.

Three design charts are presented for the overlay thickness design. Selection of the proper chart depends on the structural condition of the existing pavement and the thickness of stress relieving course. Fig. 3 is used when the pavement is intact, and no stress relieving or bond breaker is used. Fig. 4 is used when the pavement is intact and the geometric profile is such that a thin stress relieving course of 1/2 to 1 inch is used. Fig. 5 is used when the existing pavement is (a) intact but roughness or profile requires 3 inches or more of bituminous overlay, (b) the pavement has an existing bituminous overlay and no rocking slabs are presented and (c) where the pavement is in the broken condition and a 3-inch bituminous overlay is recommended. For slabs in a shattered condition, Fig. 5 may be used as a guide provided 3 or more inches of bitum-

Figure 1. Summary of concrete overlays on concrete pavements.

	UNBONDED OR SEPARATED OVERLAY	PARTIALLY BONDED OR DIRECT OVERLAY	BONDED OR MONOLITHIC OVERLAY
TYPE OF OVERLAY	T_R T_O	T_R T_O	T_R T_O
PROCEDURE	CLEAN SURFACE DEBRIS AND EXCESS JOINT SEAL PLACE SEPARATION COURSE-PLACE OVERLAY CONCRETE.	CLEAN SURFACE DEBRIS AND EXCESS JOINT SEAL AND REMOVE EXCESSIVE OIL AND RUBBER-PLACE OVERLAY CONCRETE	SCARIFY ALL LOOSE CONCRETE, CLEAN JOINTS, CLEAN AND ACID ETCH SURFACE - PLACE BONDING GROUT AND OVERLAY CONCRETE
MATCHING OF JOINTS IN OVERLAY & PAVEMENT: LOCATION	NOT NECESSARY	REQUIRED	REQUIRED
MATCHING OF JOINTS IN OVERLAY & PAVEMENT: TYPE	NOT NECESSARY	NOT NECESSARY	REQUIRED
REFLECTION OF UNDERLYING CRACKS TO BE EXPECTED	NOT NORMALLY	USUALLY	YES
REQUIREMENT FOR STEEL REINFORCEMENT	REQUIREMENT IS INDEPENDENT OF THE STEEL IN EXISTING PAVEMENT OR CONDITION OF EXISTING PAVEMENT.	REQUIREMENT IS INDEPENDENT OF THE STEEL IN EXISTING PAVEMENT STEEL MAY BE USED TO CONTROL CRACKING WHICH MAY BE CAUSED BY LIMITED NON-STRUCTURAL DEFECTS IN PAVEMENT.	NORMALLY NOT USED IN THIN OVERLAYS. IN THICKER OVERLAY STEEL MAY BE USED TO SUPPLEMENT STEEL IN EXISTING PAVEMENT
FORMULA FOR COMPUTING THICKNESS OF OVERLAY (T_r) NOTE: T IS THE THICKNESS OF MONOLITHIC PAVEMENT REQUIRED FOR THE DESIGN LOAD ON THE EXISTING SUPPORT. C IS A STRUCTURAL CONDITION FACTOR. T_R SHOULD BE BASED ON THE FLEXURAL STRENGTH OF	$T_r = \sqrt{T^2 - CT_0^2}$ OVERLAY CONCRETE	$T_r = \sqrt[1.4]{T^{1.4} - CT_0^{1.4}}$ OVERLAY CONCRETE	$T_r = T - T_0$ NOTE: THE ABILITY OF THE OVERLAID SLAB TO TRANSFER LOAD AT THE JOINTS SHOULD BE ASSESSED SEPARATELY. EXISTING CONCRETE
MINIMUM THICKNESS	6"	5"	1"
APPLICABILITY OF VARIOUS OVERLAY TYPES — STRUCTURAL CONDITION OF EXISTING PAVEMENT: NO STRUCTURAL DEFECTS C = 1.0*	YES	YES	YES
LIMITED STRUCT DEFECTS C = 0.75*	YES	ONLY IF DEFECTS CAN BE REPAIRED	ONLY IF DEFECTS CAN BE REPAIRED
SEVERE STRUCT DEFECTS C = 0.35*	YES	NO	NO
SURFACE CRACKS, SCALING, SPALLING AND SHRINKAGE CRACKS: NEGLIGIBLE	YES	YES	YES
LIMITED	YES	YES	YES
EXTENSIVE	YES	NO	YES

* C VALUES APPLY TO STRUCTURAL CONDITION ONLY, AND SHOULD NOT BE INFLUENCED BY SURFACE DEFECTS.

Figure 2. Nonrigid overlay design procedure.

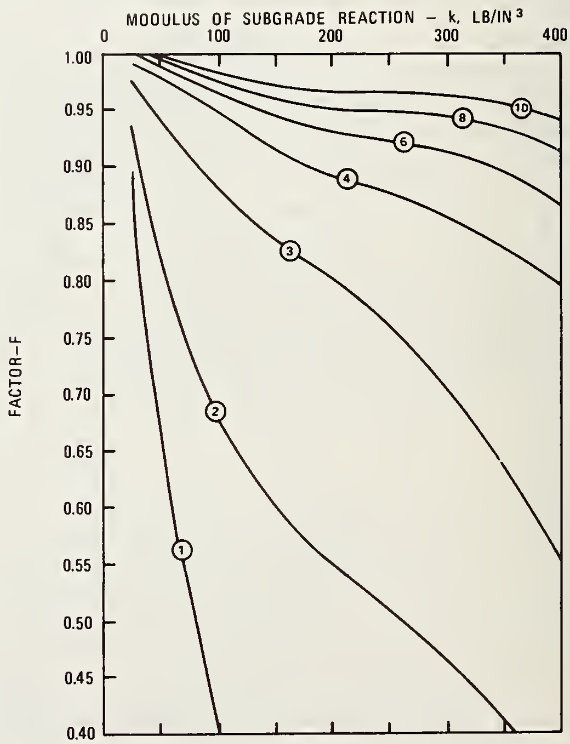


Figure 3. CRC overlay--no bond breaker.

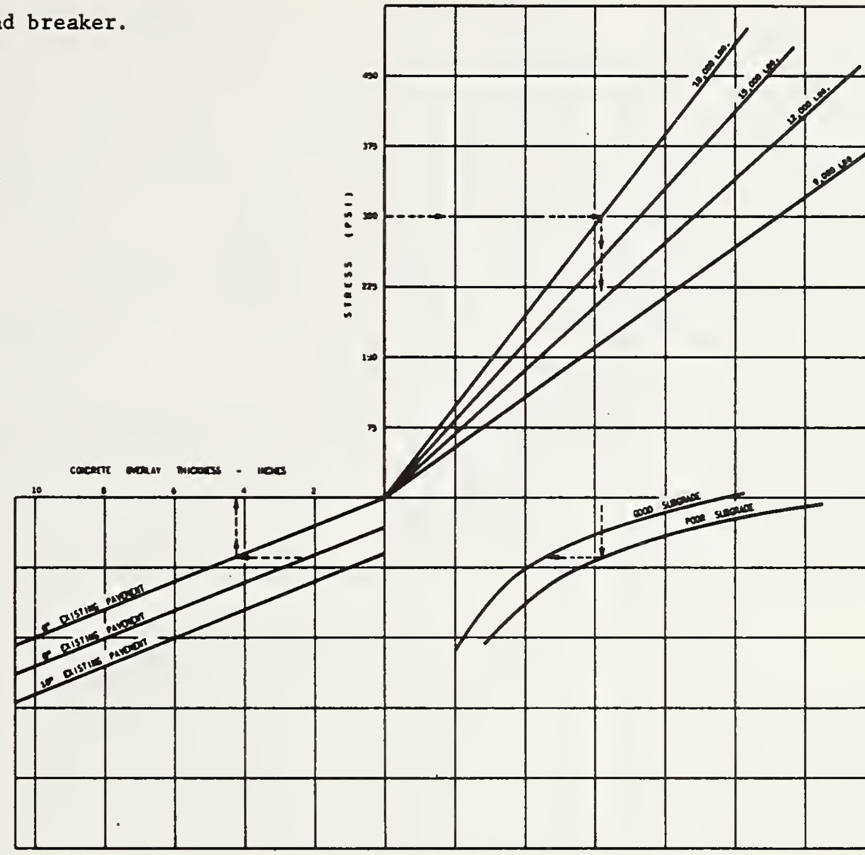
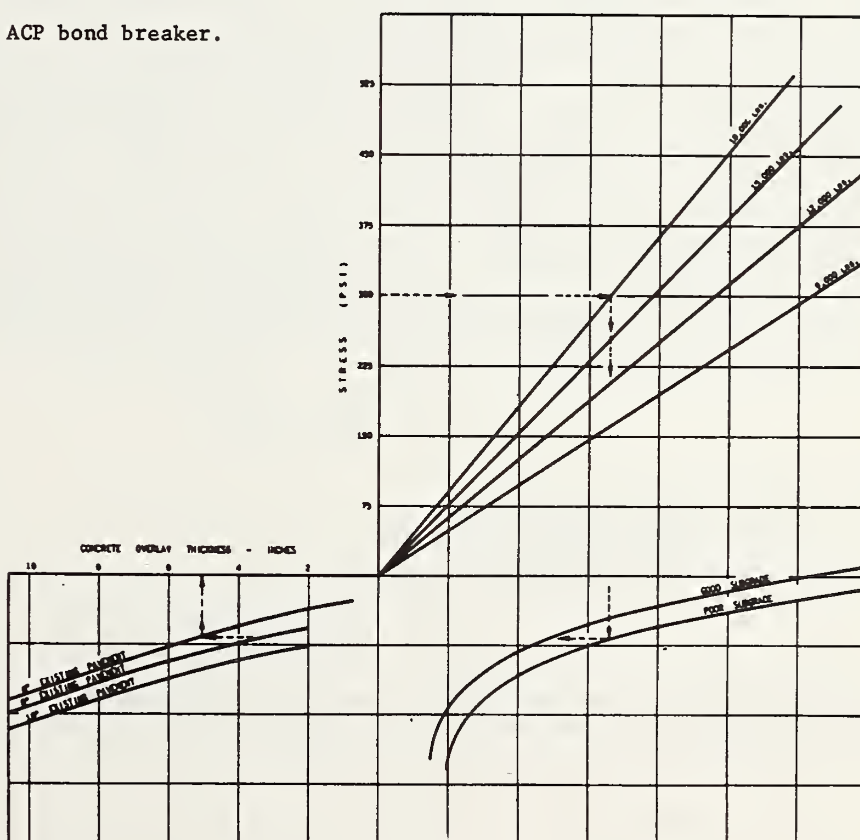


Figure 4. CRC overlay--½-inch ACP bond breaker.



inous material are used. The procedure does not deal with design of CRCP overlays over CRCP.

Maryland has recently overlaid and widened a 12-mile portion of I-70N with 6-inch CRCP overlay. A minimum of 1-inch bituminous course was placed over the old 9-inch pavement with 40-foot joint spacing. An HPR study is under way to evaluate its performance.

Attempts have been made to improve the performance of conventional CRC pavements, and there have been innovations in design of CRC resurfacings. The concept, called elastic jointed CRCP, uses weakened plane joints with continuous steel bars across the joints to form transverse hinges across the pavement, hence the name elastic joints (7, 8). The weakened plane joints may be formed by embedded strips above or below the continuous steel or by sawing. The steel is coated with a bond breaker for a short distance across the joints. With this lack of bond and crack starters, the jointed spacing can be controlled as shown in Fig. 6.

There is no field experience with elastic jointed CRCP in the United States to date, but limited experience in Europe indicates that this design may be more economical and result in better performance than conventional CRCP. The first installation of elastic jointed CRCP was placed in Green County, Iowa, and is currently being evaluated.

The use of controlled cracks or joints should eliminate the erratic crack pattern that frequently accompanies CRC pavements. The bond breaker on the steel, in the vicinity of the joint, distributes stresses in the steel over several inches of length, minimizing any tendency for steel rupture and reducing the possibility of corrosion of the steel in the joint.

The real economies for the use of elastic jointed CRCP lie in the low percentage of steel (0.2 was used in Sweden) and reduced thickness.

An ad hoc committee for concrete overlays is promoting research demonstration projects to evaluate the use of elastic jointed CRCP overlays and thinner conventional CRCP overlays.

ILLINOIS PROCEDURE FOR BITUMINOUS RESURFACING OF PCC PAVEMENT

The procedure developed by Illinois (9) is a modification of the existing design procedure. A large number of resurfaced PCC pavements were studied, and their performance was evaluated relative to performance predictions for new pavements. The analysis showed that the performance of the resurfaced pavements resembled the performance of rigid pavement more closely than it does the performance of a flexible pavement. With this finding, a design procedure was developed by modifying the currently used rigid pavement procedures. The procedure is for both first and second resurfacings, utilizing the same format as the existing procedures and the same design parameters: traffic, soil support, and material thicknesses. Traffic is evaluated in terms of equivalent 18-kip single axle load applications based on the AASHO Road Test performance equation for rigid pavements. Material thicknesses are included in a linear, structural number relationship similar to that employed in the Illinois and AASHO Interim Guide procedures for flexible pavements. The design analysis is presented in the form of nomographs that include the three design parameters.

Equivalent 18-kip single axle load applications (traffic factor) are determined by using the appropriate equation from Fig. 7, a selected design period not exceeding 15 years, and the estimated average daily traffic for the middle year of the design period. The next step in the state's procedure is to determine the soil support value. This is usually determined from original soil surveys or from construction or design records. Once the traffic factor and soil suport values have been determined, the next step is to apply them to the appropriate nomograph (Figs. 8 or 9). It is noted that the roadway classifications, defined in Fig. 7, determine which traffic factor equation and nomograph are applicable. A straight line is constructed that connects the traffic factor and soil support value on the appropriate nomograph. The point of intersection on the structural number scale is the required resurfacing structural number. The overlay thickness can then be determined by using the thickness of the existing pavement components. If it is the first overlay to be applied to the pavement, the thickness is based on

$$D_F = \frac{SNr - 0.26\ Dc}{0.40}$$

where

D_F = overlay thickness,

SNr = resurfacing structural number, and

Dc = PCC pavement thickness.

For second overlays the thickness is determined by

$$Ds = \frac{SNr - (0.25\ D_F + 0.17\ Dc)}{0.40}$$

where Ds is the second overlay thickness. However, it is suggested by Illinois that, for Class I and Class II roads, the overlay thickness be a minimum of 3 inches and, for Class III and Class IV

Figure 5. CRC overlay--3-inch ACP bond breaker.

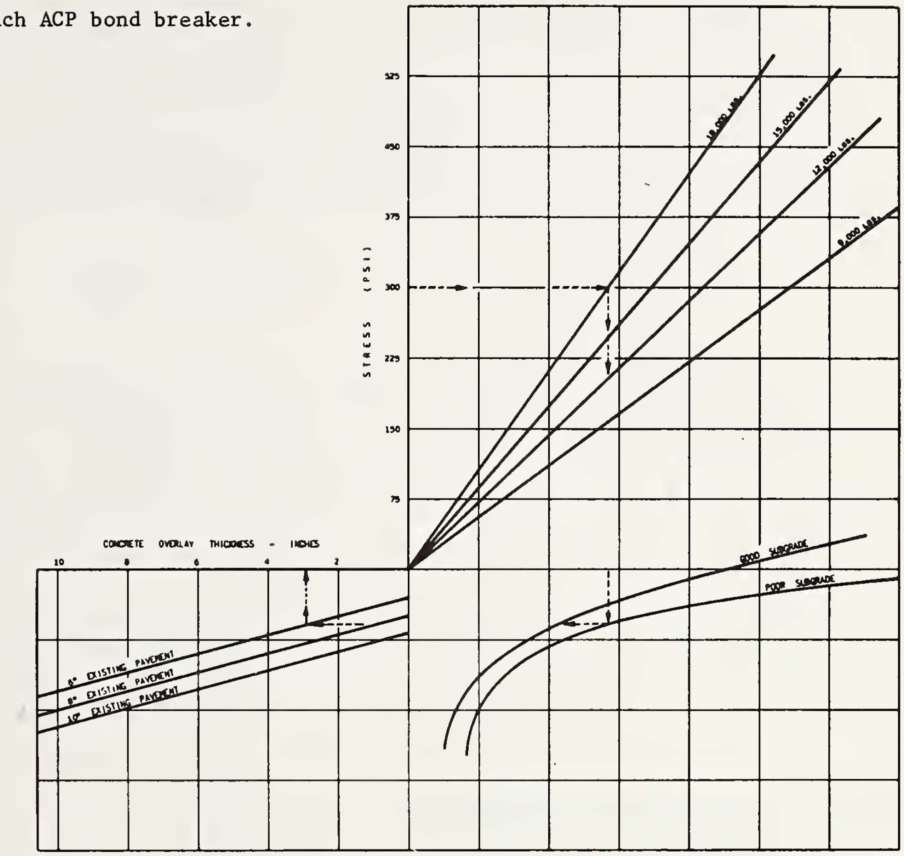


Figure 6. CRC overlay with elastic joints.

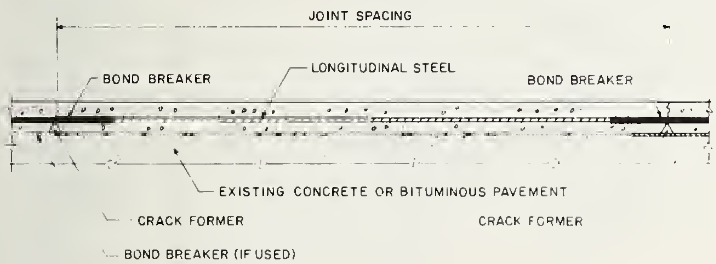


Figure 7. Definitions of road classes and traffic equations.

DEFINITIONS

<u>Class I Roads and Streets</u>

Trunk, major, area service, and collector roads and streets designed as four-lane or more facilities, and one-way streets with structural design traffic greater than 3,500 ADT.

<u>Class II Roads and Streets</u>

Major and area service roads and streets designed as two-lane facilities, one-way streets with structural design traffic less than 3,500 ADT and collector routes designed as two-lane facilities with structural design traffic greater than 2,000 ADT.

<u>Class III Roads and Streets</u>

Collector routes designed as two-lane facilities with structural design traffic between 750 and 2,000 ADT.

<u>Class IV Roads and Streets</u>

Collector and land access routes with structural design traffic less than 750 ADT.

TRAFFIC FACTOR EQUATIONS

<u>Class I Roads and Streets</u>

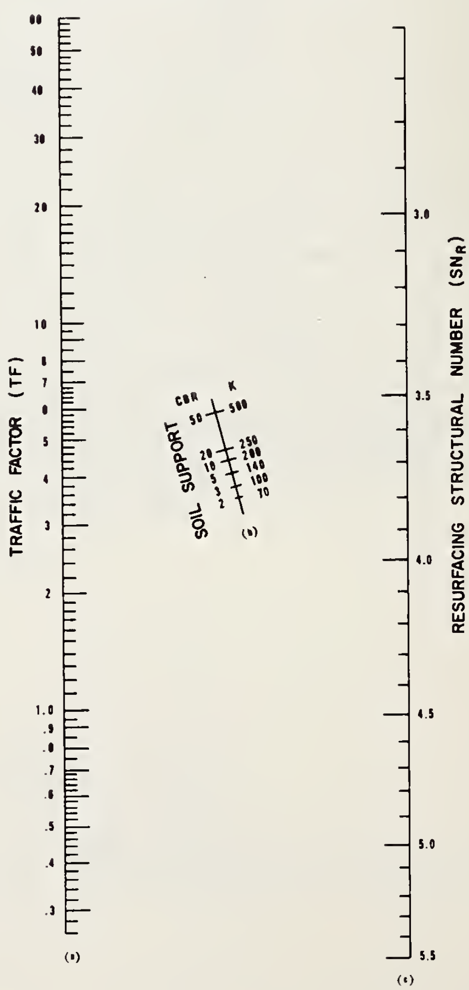
$$TF = \frac{DP(0.146 \times U_P \times PC + 44.895 \times U_S \times SU + 421.575 \times U_M \times MU)}{1,000,000}$$

<u>Class II, III, and IV Roads and Streets</u>

$$TF = \frac{DP(0.146 \times U_P \times PC + 44.895 \times U_S \times SU + 413.910 \times U_M \times MU)}{1,000,000}$$

where:
- TF = equivalent 18-kip single axle load applications in millions
- DP = design period, not to exceed 15 years
- PC = average daily passenger car traffic
- SU = average daily single unit traffic
- MU = average daily multiple unit traffic
- U_P = percent passenger cars in design lane
- U_S = percent single units in design lane
- U_M = percent multiple units in design lane

Figure 8. Nomograph for resurfacing PCC pavements--class I roads (PSI = 2.5).



roads, a minimum of 2-1/4 and 1 - 1/2 inches respectively.

The procedures as developed by Illinois provide a means of determining the resurfacing thickness required to upgrade structural capabilities and restore the riding quality of properly repaired existing concrete pavements and resurfaced concrete pavements that have already served their original lives. It is not intended for use in selecting thicknesses for structurally sound pavements that require resurfacing simply to restore riding qualities lost because of scaling, raveling, or inadequate skid resistance. Also, because the procedure was developed with the assumption that the existing pavement will be properly repaired prior to placement of the resurfacing, the selected thickness will not be adequate to bridge highly distressed or failed areas in the existing pavement.

CALIFORNIA

Historical Experience

The California Division of Highways has long been a leader in pavement research and in the development of overlay design procedures for flexible pavements (10). It has a well documented overlay design method based on deflections in Test Method Calif. 356 with the latest revision issued October 1973 (11).

California's experience with deflection measurements dates back to 1938. The earliest device used for measuring pavement deflection was the General Electric Travel Gage. This instrument was installed on various California highways as early as 1938 and on the Brighton Test Track in 1940 and later, during World II, on the Stockton Test Track. The installation of these units required the drilling of 5-inch diameter holes through the pavement surface and the insertion of rods to depths of up to 18 feet into the pavement section. Through installations at various depths it was possible to measure not only the total deflection but also the compression contributed by each element of the structural section. It was found that pavement deflection was measurable up to depths of about 21 feet. However, the majority of deflection occurred in the top 3 feet of the structural section.

Because the use of General Electric Travel Gage units was expensive from an installation standpoint and relatively few measurements could be made per day, the need arose for a better deflection measuring device. An improved version, using the linear variable differential transformer (LVDT), was tried at the WASHO Road Test, but difficulties in maintaining calibration persisted. It was not until 1952 when Benkelman, former BPR employee, and then flexible pavement research engineer for the WASHO Road Test, made it possible to accurately measure pavement deflections with the development of his Benkelman beam. Since then, many highway organizations throughout the world have been using this simple cantilever device.

In 1954 the California Division of Highways began using the Benkelman beam. While it greatly simplified the task of measuring pavement deflection under wheel loadings, it was, however, still a manually operated, slow, and tedious process. To overcome these difficulties, an automatic deflection measuring device known as the California Traveling Deflectometer was later developed by the Materials and Research Department of the California Division of Highways and put into operation in 1960. A newer version was introduced in 1967.

Based on the Benkelman beam principle, the Deflectometer combines a truck-trailer unit that carries an 18-kip single axle load on the rear tires and a carriage to support probes for measuring pavement deflection under both wheels simultaneously. It is an electromechanical instrument that measures pavement deflection at 20-foot intervals while the vehicle moves steadily along the road at 1/2 mph. The deflections are measured to the nearest 0.001 inch by means of a probe arm resting on the pavement and are permanently recorded on chart paper. Between 1,500 and 2,000 individual deflection measurements are possible per day as opposed to about 300 measurements using the manually operated Benkelman beam.

Development of Overlay Design Method

To effectively use deflection measurements required that the magnitude of pavement deflection be related to pavement performance. This is not possible through current accelerated test track wheel loading facilities because relatively short-duration testing does not permit the average asphalt concrete surfacing to weather and harden. To establish a reasonable tie between fatigue failure of asphalt concrete surfacing and magnitude of transient deflection, it was necessary to obtain deflection measurements over roadways that had been in operation for several years. This allowed the asphalt concrete surfacing to reach a realistic or near critical state of hardness.

In 1951 a comprehensive deflection research program was initiated by California to evaluate this relationship as a primary objective. For this study General Electric Gage units were installed on 43 projects throughout California. The test roadways included a wide variety of pavement structural sections since the thickness of AC surfacing was a prime variable. Installations were made on both cracked and uncracked pavements. The results of this study were reported in 1955 by Hveem (12).

Evaluation of data from this study also suggested maximum tolerable deflection levels for various pavement thicknesses. These values represented the highest levels of transient pavement deflection that a particular pavement thickness could withstand during its design life without developing fatigue cracking. The deflection criteria, reported in 1955, provided the basis for further study inasmuch as the roads investigated were main-line pavements with relatively high traffic volumes. To be more representative of lower traffic situations, it was necessary to adjust these criteria for variations in traffic volume. This was accomplished with fatigue tests on speciments cut from various AC pavements. The criterion for tolerable deflection was adjusted accordingly. However, these criteria were tentative because the slope lines are based solely on laboratory surface fatigue data and had to be correlated with field performance. Because deflection experience was limited for lightly traveled roadways, a maximum level of 0.040 inch of tolerable deflection was suggested. A change in asphalt specifications was made in 1960 in hopes of producing more durable AC pavements and may change the tolerable limits in the future.

California is conducting a research project to evaluate present deflection criteria by relating pavement performance to tolerable deflection level, structural section, asphalt hardness properties, and traffic loading. Deflection attenuation properties of various roadway materials are also being investigated on reconstructed highway projects based on California's present overlay design method using pavement deflection measurements. The California overlay design procedure has been revised as a result of this research.

Overlay Design Procedure

The latest California overlay design procedure is in Test Method Calif. 356-D dated October 1973. The latest revision was brought about by updating the old procedures with additional data being obtained on various overlay projects under study.

The test method describes four pavement deflection measuring devices and the procedures used for determining overlay requirements for existing AC roadways by deflection analyses. The method consists of measuring the total pavement deflection resulting from the application of an 18-kip single axle load (9-kip dual wheel load). The deflection readings are then compared to allowable limits for a similar structural section and traffic volume. Corrective treatment is described as the overlay required to reduce the deflection to a level at which the surface will be unlikely to fail due to fatigue. The measurement of deflection can be made by either the Benkelman beam, the Traveling Deflectometer, the Dynaflect, or the Dehlen curvature meter.

Field work consists of selecting representative test sections for each change in visual condition or known change in structural section. Each test section should be referenced to a known or easily identifiable point in the field. As a safety precaution, all test sections should include sufficient sight distance in both directions. Therefore the location of test sections on horizontal or vertical curves should be avoided. Each test section should normally vary from 800 to 1,000 feet in length and represent a centerline mile of roadway.

For Benkelman beam, the WASHO method is used. Readings are taken at 25-foot intervals, alternating two measurements in the outer wheel track for every one in the inner wheel track throughout the test section. With the Traveling Deflectometer, measurements are made in both wheel tracks at 20-foot intervals at a travel speed of 1/2 mph, allowing the operator to note unusual pavement features such as patches, poor drainage, and cracking. Some disadvantages of the Deflectometer are the high initial cost, the high maintenance cost, and its inability to operate properly on short radius curves.

For the Dynaflect, maximum deflection values are obtained every 0.01 mile on the wheel track exhibiting the most distress, with a minimum of 20 measurements per test section. Conversion to Traveling Deflectometer deflections is made by using Fig. 10. Rough estimates of deflection values are obtained by making curvature readings at 25-foot intervals, alternating two measurements in the outer wheel track for every one in the inner wheel track as with the Benkelman beam. Radius of curvature in feet is computed and converted to equivalent Traveling Deflectometer deflection levels using Fig. 11 or 12, depending on whether the existing structural section consists of untreated aggregate base or cement treated base. While the Dynaflect has good mobility and is very effective on city streets where selective locations are necessary because of utility trenches, on high-speed highways its use requires lane closures. It operates at only one frequency and makes fewer measurements per day on rural highways than the Deflectometer.

For all measurements, the average (mean) and the evaluated 80th percentile deflection levels are calculated and reported. The evaluated 80th percentile indicates that 20 percent of the readings are higher and 80 percent lower.

Once the 80th percentile deflection is obtained, the analysis of the data and selection of the overlay are relatively straightforward. The procedure states that for an effective overlay design the following factors must be considered: cause of pavement failure, existing structural section

Figure 9. Nomograph for resurfacing PCC pavements--classes II, III, and IV roads (PSI = 2.0).

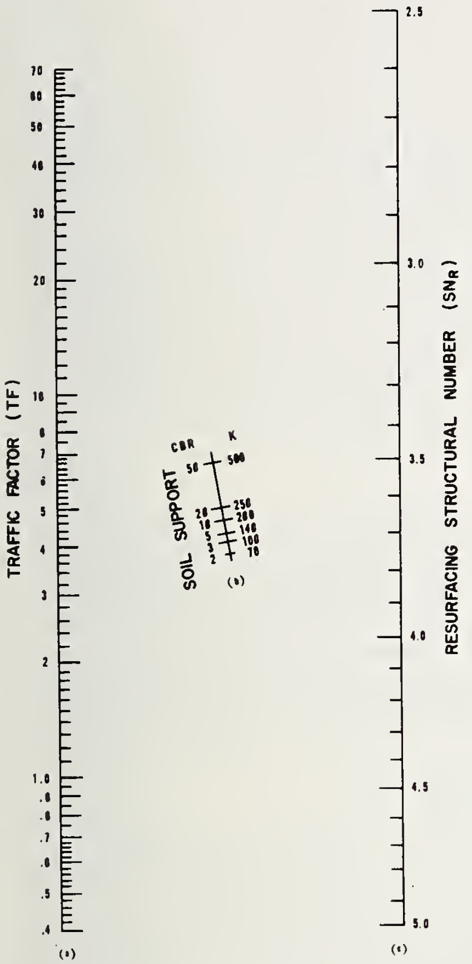


Figure 10. Comparison of Dynaflect and traveling deflectometer.

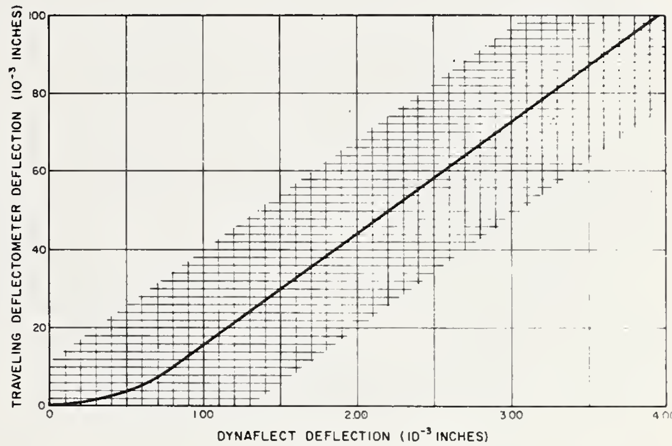


Figure 11. Radius of curvature versus deflection for asphalt concrete over aggregate base.

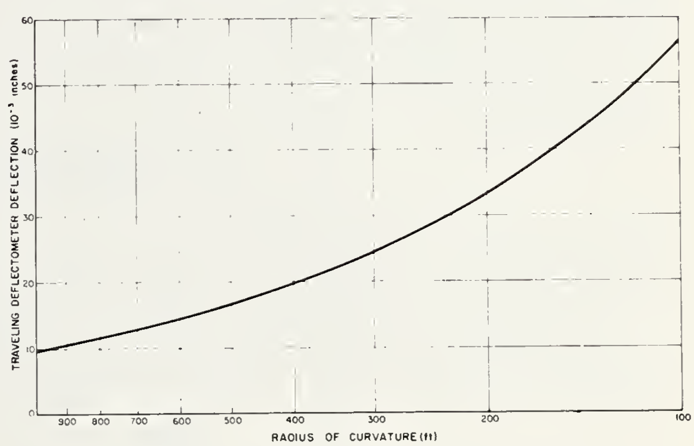
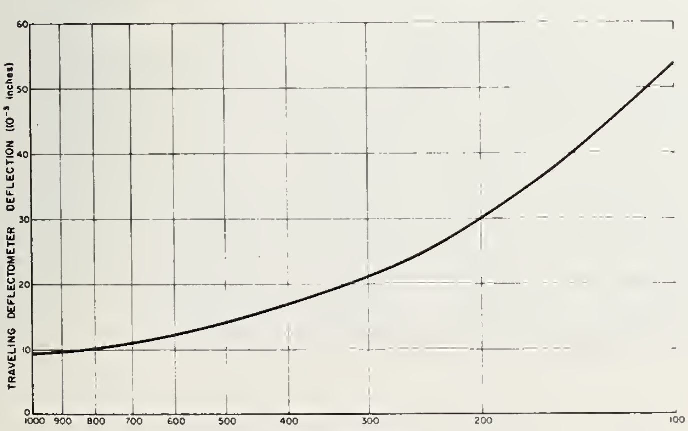


Figure 12. Radius of curvature versus deflection for asphalt concrete over cement-treated base.



materials, deflection magnitude of existing section, reflection cracking potential, traffic, and tolerable deflection level. Each is considered in the procedure.

The concept of an allowable or tolerable level of deflection as related to total traffic and existing structural section is the backbone of the California procedure. The latest revised tolerable deflection chart is shown in Fig. 13. The next step is to compare the measured evaluated 80th percentile deflection to the allowable deflection level for the existing pavement thickness at the design traffic as determined from Fig. 13. If the measured evaluated 80th percentile deflection is less than the allowable deflection and reflection cracking does not occur, no corrective repair is necessary other than a seal coat or thin AC blanket to seal cracks or improve appearance and riding quality. If the measured evaluated 80th percentile deflection is more than the allowable deflection, the overlay design chart (Fig. 14) is used. Enter the chart with the predicted traffic, and follow this value vertically to the deflection curve corresponding to the 80th percentile deflection. The thickness required is read off the vertical scale.

For some pavements, the magnitude of the existing deflection level is not a governing criterion for design. Frequently, the need to eliminate potential reflection cracking from the underlying pavement establishes the AC blanket thickness. At present, there is no set method to determine this thickness, but a rule of thumb generally used for prevention of reflection cracking is as follows: The new blanket thickness should be at least half the thickness of the existing AC pavement over untreated bases. For PCC pavements or existing AC pavements over CTB, a minimum overlay thickness of 0.30 foot should be used. A lesser overlay thickness could provide a smooth riding surface but will allow existing cracks to reflect through the overlay prematurely.

Previous Method

Because of California's leadership in overlay design, it is important to look at previous design procedures and the changes that have been made with time. Test Method 356 was originally issued in 1969 with revisions in 1970 and 1972. The procedure remained essentially the same with these revisions. The measurement of deflection, data collection, and reporting of the evaluated 80th percentile Traveling Deflectometer deflection levels are the same as previously stated. The measured evaluated 80th percentile deflection is compared to the allowable deflection level for the existing pavement thickness at the design traffic. If the measured evaluated 80th percentile deflection is greater than the allowable deflection, the required percentage of reduction in evaluated deflection level is calculated by using the tolerable deflection for the thickness of proposed surfacing.

$$\text{Deflection reduction (\%)} = \frac{\text{80th percentile level} - \text{tolerable level}}{\text{80th percentile level (100)}}$$

With this value of required reduction, Fig. 15 is entered to find the necessary cover required in terms of gravel equivalent (GE). This GE is then compared with the GE of the tentatively selected overlay thickness. Several trials may be necessary to arrive at a satisfactory design.

Under certain circumstances it was possible not to reach an end point. Each overlay increased the thickness of the pavement, which, for a given traffic, reduced the tolerable deflection, in turn requiring a thicker overlay to reduce the deflection. The 1973 design procedure has been changed to virtually eliminate this possibility in the design process.

ASPHALT INSTITUTE METHOD

The Asphalt Institute's overlay design procedures are contained in the Asphalt Institute Manual Series 17 (MS-17) (13). Three methods of overlay are presented, one for overlays over PCC pavements and two for overlays of asphalt pavement structures.

Overlay Design by Deflection Analysis

This method is used for designing AC overlays for asphalt pavements. It is based on elastic-layered theory and engineering experience with the use of deflections. The method requires an evaluation of the existing pavement to determine its strength and the traffic expected to use the facility. The assumptions associated with the procedure are: (a) the higher the level of pavement deflection is, the shorter the time will be until the pavement requires an overlay, (b) tolerable deflection is a function of traffic, and (c) the additional thickness of AC on an existing pavement will reduce the deflection to an acceptable level. In this procedure pavement deflection is measured with a Benkelman beam and the representative rebound deflection is determined. By using this value and the design traffic number (DTN), the AC overlay thickness is determined from Fig. 16. By entering the deflection on the abscissa of the chart and moving vertically to the design DTN curve (interpolate if necessary), the overlay thickness is read on the ordinate.

The rebound test procedure, using an 18-kip single axle load, is used with the Benkelman beam

Figure 13. Tolerance deflection chart for varying thicknesses of asphalt concrete.

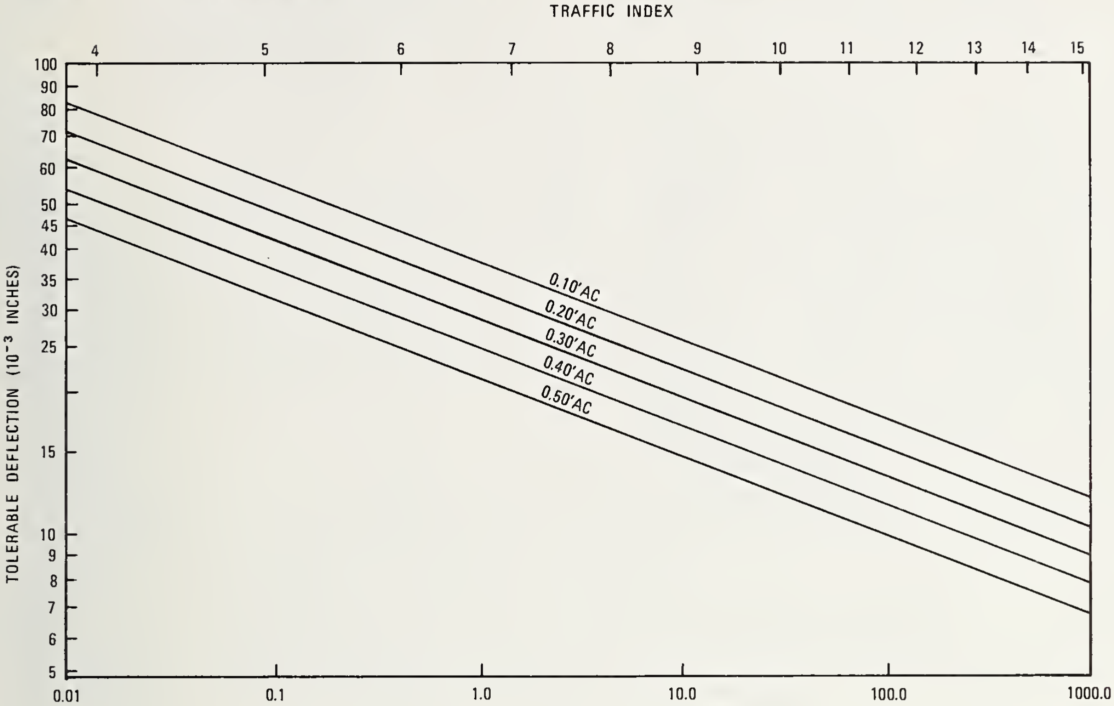
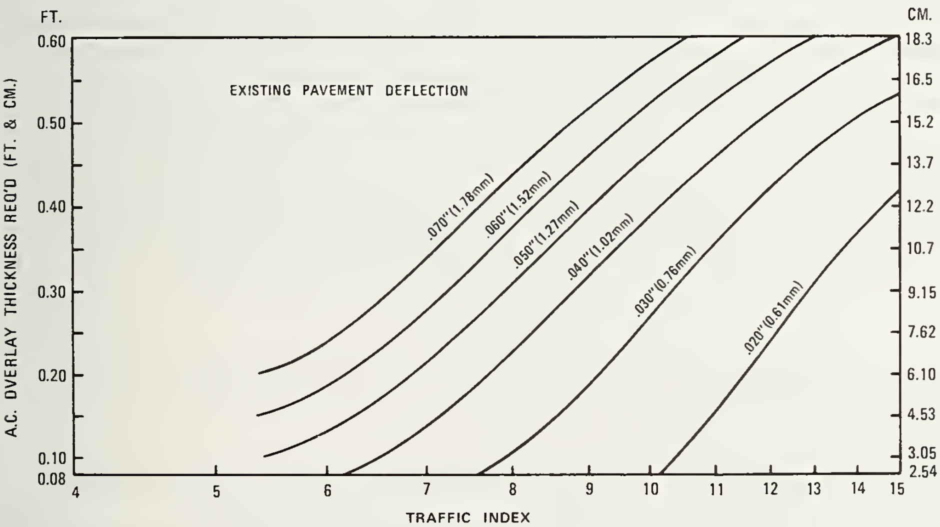


Figure 14. AC overlay design guide.



to measure deflections. The measured deflections are reduced to a representative rebound deflection, which is defined as the mean of the adjusted measured rebound deflections plus two standard deviations. Measured rebound deflections should be adjusted for temperature and the most critical period of the year for pavement performance. The representative rebound deflection equals $(\bar{X} + 2s)fc$ where $\bar{X}$ = arithmetic mean, s = standard deviation, f = temperature adjustment factor, and c = critical period adjustment factor (c = 1 for most critical period). Detailed procedures for arriving at each of these values are given in MS-17.

DTN, as used by the Asphalt Institute, is defined as the average daily number of equivalent 18-kip single axle load applications expected for the design lane during the design period. For overlay design, the design period is the number of years until the overlay is resurfaced. Methods of determining DTN are given in Asphalt Institute publications MS-17 and MS-1 (14). Kingham (15), in his derivation of the overlay design, has stated three principles inherent in the procedure. The first is that, for a given material, the higher the level of pavement deflection is, the shorter the pavement life will be. The second states that a tolerable deflection is a function of traffic and the third that additional thicknesses of asphalt concrete placed on the existing pavement will reduce the deflection, and, if sufficient thickness is added, the deflection will be reduced to a tolerable level. These principles suggest that a design method can be based on two relationships. The first relates tolerable deflection level to traffic while the second relates reduction in deflection to a thickness increase.

The traffic deflection relationship was established by examining a great deal of experience from various agencies. It involved looking at the WASHO and AASHO Road Test data, Benkelman's results, and experience in California, Canada, and several foreign countries. The experience of the various agencies is shown in Fig. 17. The Asphalt Institute adopted the heavy dark line in the figure as its design line. This is a conservative approach and, with it, the probability of an unsatisfactory design is very low. It should be noted that there is a gray area in the use of deflection criteria and judgment should be used in the selection of the design criteria. Furthermore, it should be stated that many different deflection measuring devices are used by the various agencies, ranging from light to heavy loadings, which may be either statically or dynamically applied. Users of these systems should be aware that the basic empirical cures for converting deflection to thickness of overlay are only valid when the measurements used are compatible with the curves.

Kingham states, "The assumption of a tolerable deflection related to future traffic use is not a wholly satisfactory failure criterion in terms of engineering mechanics. It is obvious that a traffic-deflection relationship will be different for different materials. However, considerable experience has been gained with pavement deflection on pavements having asphalt surfaces and granular bases, and this suggests that for this type of construction there is a relationship that can be put to practical use. Deflection is admittedly a crude parameter compared to more sophisticated parameters such as strain and stress, but it appeals to engineers because it is easily measured."

In establishing the thickness deflection relationship, the Asphalt Institute uses one of the major findings published at the 1967 Ann Arbor conference that elastic layered theory can be used to predict deflections (16).

An assumption used in the procedure is that the existing pavement and subgrade can be represented by an effective modulus, E_s. This modulus represents the foundation support to the overlay and is derived from the representative deflection by use of the Boussinesq equation:

$$d_s = \frac{1.5\,pa}{E_s}$$

where

- d_s = representative pavement deflection (inches),
- p = constant pressure, 70 psi,
- a = radius of single plate, 6.4 inches, and
- E_s = effective modulus.

This assumption has been made by numerous investigators and has been found to give a reasonable approximation to multilayer elastic theory.

The thickness of overlay required to reduce the representative deflection to a tolerable deflection can then be calculated from elastic layer theory. The Asphalt Institute used the two-layer relationship developed by Kirk (17) given below:

$$d = \frac{1.5\,pa}{E_s}\left[\left(1 - \frac{1}{\sqrt{1 + 0.8\left(\frac{t}{a}\right)^2}}\right)\frac{E_s}{E_p} + \frac{1}{\sqrt{1 + \left(0.8\,\frac{t}{a}\sqrt[3]{\frac{E_p}{E_s}}\right)^2}}\right]$$

Figure 15. Reduction in deflection resulting from pavement reconstruction.

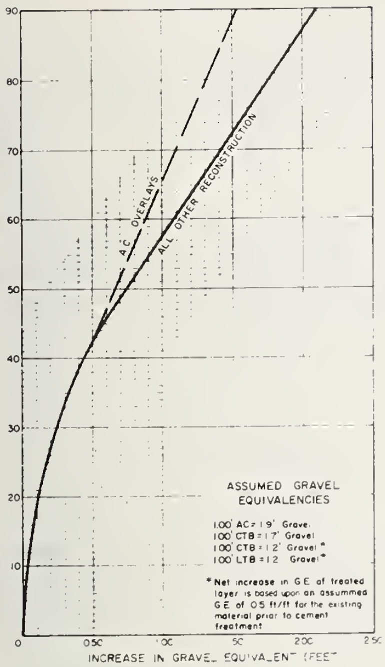


Figure 16. Thickness design chart.

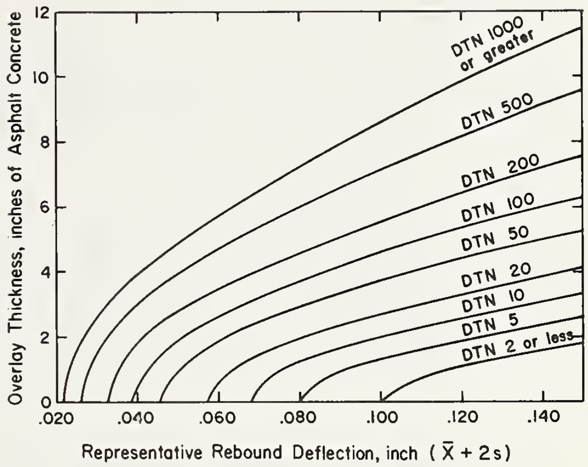
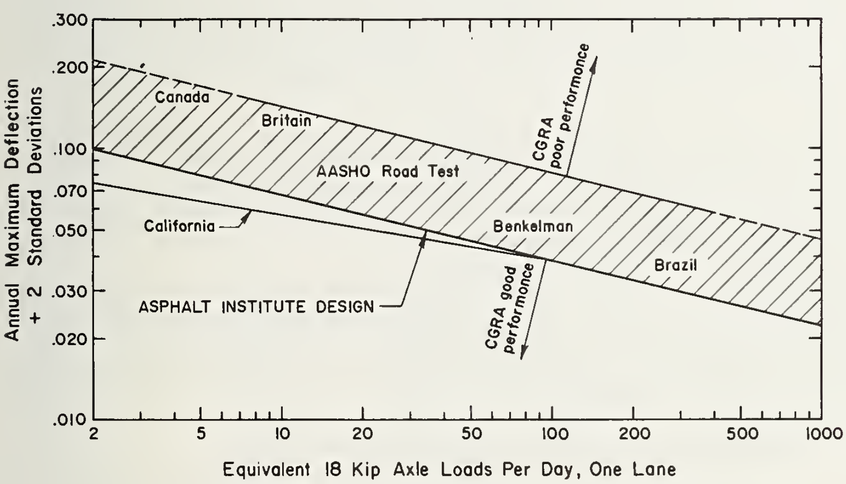


Figure 17. Compilation of deflection experience.



where

d_t = tolerable pavement de-lection (inches),

t = overlay thickness (inches), and

E_p = overlay modulus.

The design curves in Fig. 16 were determined by solving this equation for t using various values of d_t and E_s, which relate to design traffic and representative deflection as previously stated. The modulus of the AC overlay was assumed to be 500,000 psi, which Kingham states is typical for asphalt concrete tested at 1 cps on a cylindrical specimen at 70 F.

Overlay Design by Component Analysis

The procedures for designing strengthening overlays by component analysis over AC and PCC pavements by the Asphalt Institute's method are modifications of existing designs. The procedure is based on the presumption that the old and new layers will form a composite pavement structure with the needed strength. The old pavement structure must be evaluated on the basis of the quality and thickness of its components before the overlay design can be computed.

The thickness of overlay is based on the equation

$$T_o = T - T_e$$

where

T_o = thickness of overlay,

T = thickness of pavement structure required by conventional pavement analysis, and

T_e = effective thickness of the existing pavement structure.

Factors used for correcting existing thickness of the pavement components to the effective thickness have been arranged to consider the condition of the material and the loss of load carrying capability as shown in Fig. 18. Evaluating the condition of the layers requires a subjective and experienced observer. Once the component layers are identified and the conversion factors are determined, the effective thickness of each layer is calculated and summed to give T_e of the total pavement.

The traffic analysis consists of determining the DTN and is the same procedure used in the deflection method. Subgrade analysis procedures are outlined in the Asphalt Institute's Soils Manual (MS-10). Soaked CBR, plate-bearing values, or R-values may be used.

The total thickness of pavement required is then obtained from either Fig. 19 or 20. From this value, the effective thickness is subtracted to give the thickness of overlay.

This procedure applies to overlays over both AC and PCC pavements. With rigid pavements, a minimum thickness of asphalt overlay of 4.5 inches is recommended to prevent reflection cracking. This is increased to 7 inches for weak subgrades. It is also pointed out that the most effective way to minimize reflection cracking is to break the slabs into small pieces about 2 feet across and seat them firmly. This, however, reduces the thickness conversion factor to that of granular base (Fig. 18).

A method for estimating pavement life, in terms of the estimated number of years before an overlay is needed, is detailed in the Asphalt Institute procedure. The design subgrade strength value and the effective thickness (T_e) is determined as outlined previously. Fig. 19 or 20 is entered letting T_e = total thickness, and the DTN is determined. By knowing the design traffic and the initial traffic (existing traffic), the traffic analysis procedure is worked backward to yield the design period or how long before an overlay is needed.

This method is an indicator only and is not to be used as an unfailing predictor. The manual states, "Too many forces are constantly working within the pavement for any method for estimating its life to be considered absolutely accurate. The estimate, however, is a valuable tool to use in planning future work, especially if the pavement section is reevaluated every two or three years to check the estimate and establish the trend of performance."

ROAD AND TRANSPORTATION ASSOCIATION OF CANADA METHOD

The overlay design procedure of the Road and Transportation Association of Canada (RTAC), formerly the Canadian Good Roads Association (CGRA), is similar to the Asphalt Institute method (18). It is based on considerable experience with deflections. The life of some existing flexible pavements may be extended by providing an overlay of sufficient thickness to reduce their rebound deflection value to an acceptable level that will ensure adequate supporting capacity. Factors weighed in making a decision on whether an overlay is needed are Benkelman beam deflection, a riding comfort index, a condition survey, and traffic and maintenance costs.

The additional thicknesses, expressed in inches of granular base, required to reduce the Benkelman beam rebound value ($\bar{X} + 2\sigma_x$) of a section of existing flexible pavement to values of 0.020, 0.030, 0.040, and 0.050 inch, are given in Fig. 21. Once the rebound value ($\bar{X} + 2\sigma_x$) for a section of flexible pavement is known, the additional thickness of granular base required to strengthen

Figure 18. Thickness requirements using CBR or plate-bearing values.

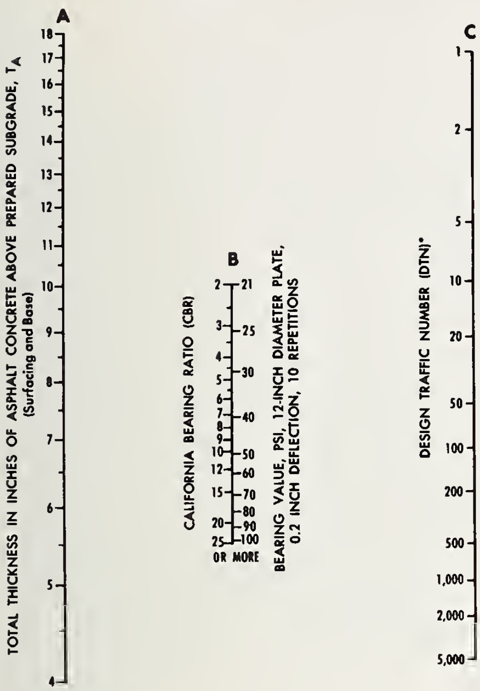


Figure 19. Thickness requirements using R-value.

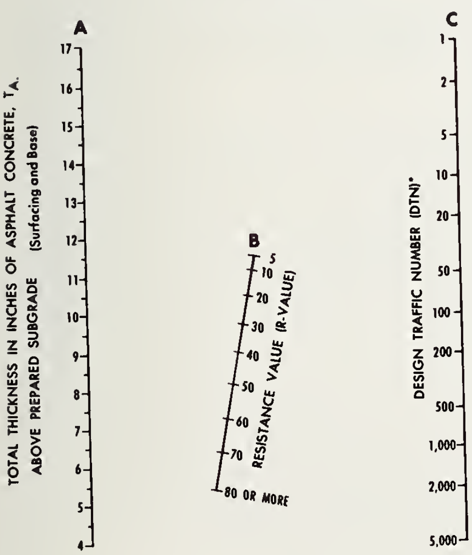
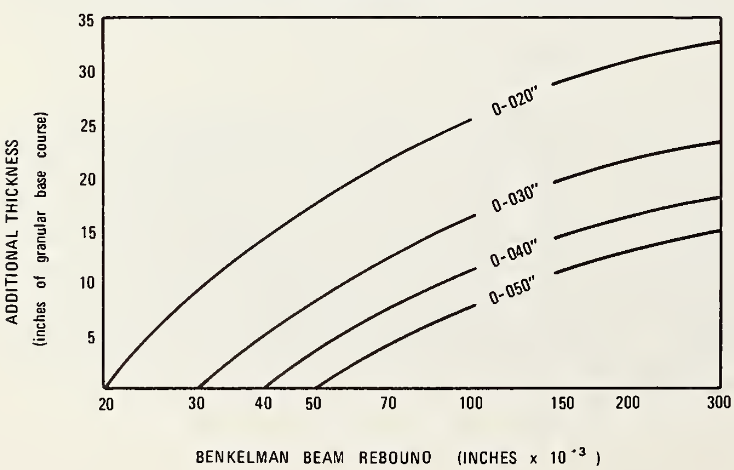


Figure 20. Factors for converting existing thickness to effective thickness.

Classification	Description of Material	Conversion Factors[a]
I	Native subgrade in all cases.	0.0
II	(a) Improved subgrade constructed with predominantly granular materials that may contain some silt and clay but have PI of 10 or less. Improved subgrade is any course or courses of improved material between the native subgrade soil and the pavement structure. (b) Lime-modified subgrade constructed from high-plasticity soils having a PI greater than 10. Lime-modified subgrade is a prepared and mechanically compacted unhardened or semihardened intimate mixture of lime, water, and soil below the pavement system.[c]	0.0 to 0.2
III	(a) Granular subbase or base constructed with reasonably well-graded, hard aggregates having some plastic fines and CBR not less than 20. Upper part of range is used if PI is 6 or less; lower lower part of range is used if PI is more than 6. (b) Cement-modified subbases and bases constructed from low plasticity soils that have a PI of 10 or less. Cement-modified subbase is an unhardened or semihardened intimate mixture of pulverized soil, portland cement, and water used as a layer in a pavement system between the subgrade and the base course. Cement-modified base is an unhardened or semihardened intimate mixture of pulverized soil, portland cement, and water and is used as a layer in the pavement system to reinforce and prtect the subgrade or subbase.[c]	0.2 to 0.3
IV	(a) Granular base constructed with a nonplastic granular material complying with established standards for high-quality aggregate base. Upper part of range is used. (b) Asphalt surface mixtures that have large, well-defined crack patterns, spalling along the cracks, and appreciable deformation in the wheel paths showing some evidence of instability. (c) Portland cement concrete pavement that has been broken into small pieces, 2 ft or less in maximum dimension, prior to overlay construction. Upper part of range is used when subbase is present; lower part of range is used when slab is on subgrade. (d) Soil-cement bases that have developed extensive crack patterns as shown by reflected surface cracks, and may exhibit pumping; pavement shows minor evidence of instability. Soil-cement base is a hardened material formed by curing a mechanically compacted intimate mixture of pulverized soil, portland cement, and water and is used as a layer in a pavement system to reinforce and protect the subgrade or subbase.[c]	0.3 to 0.5
V	(a) Asphalt surfaces and underlying asphalt bases[b] that exhibit appreciable cracking and crack patterns, have little or no spalling along the cracks, and remain essentially stable even though exhibiting some wheel path deformation. (b) Appreciably cracked and faulted portland cement concrete pavement that cannot be effectively undersealed. Slab fragments, ranging in size from approximately 1 to 4 sq yd, are well seated on the subgrade by heavy pneumatic rolling. (c) Soil-cement bases that exhibit little cracking, as shown by reflected surface crack patterns, and that are under stable surfaces. (See definition of soil-cement base under IV-d of this table.)	0.5 to 0.7
VI	(a) Asphalt concrete surfaces that exhibit some fine cracking, small intermittent cracking patterns, and slight deformation in the wheel paths though they remain stable. (b) Liquid asphalt mixtures that are stable, generally uncracked, show no bleeding, and exhibit little deformation in the wheel paths. (c) Asphalt treated base, other than asphalt concrete.[b] (d) Portland cement concrete pavement that is stable and undersealed, has some cracking, but contains no pieces smaller than about 1 sq yd.	0.7 to 0.9
VII	(a) Asphalt concrete, including asphalt concrete base, that is generally uncracked and has little deformation in the wheel paths. (b) Portland cement concrete pavement that is stable, undersealed, and generally uncracked. (c) Portland cement concrete base, under asphalt surface, that is stable, nonpumping, and exhibits little reflected surface cracking.	0.9 to 1.0

Figure 21. Depths of granular base required to reduce deflection to designated rebound value.



it to a design rebound value can be determined from the appropriate curve of Fig. 21. This thickness of granular material may then be converted to a practical overlay design consisting of an AC surface with or without a layer of granular base.

The principal steps in the flexible pavement overlay design procedure are as follows:

1. Divide the pavement into sections on which the overlay thicknesses are to be determined separately. The design sections shall be either 1,000 feet long, or of variable lengths determined on the basis of changes in subgrade soil, existing pavement performance, thickness, or construction and maintenance standards.
2. Estimate the maximum spring Benkelman beam rebound value ($\overline{X} + 2\sigma_x$) for each section. The spring rebound value for each section may be determined by one of the following methods in order of preference: Make the rebound measurements on the sections in spring when they are at their peak values; make the rebound measurements at any time during the summer or fall (June to October) and convert to maximum spring values by using a continuous record of measured rebound values for a local control section having a similar subgrade soil types; or make the rebound measurements in the fall (September 1 to October 15) and convert to maximum spring values by using a ratio of 2.5.
3. Select a desired spring design rebound value between 0.030 and 0.50 inch. The choice will depend on the relative costs of initial construction and resurfacing.
4. Using the curve in Fig. 21 for the design rebound value and the estimated maximum spring rebound values, determine the additional thicknesses, in inches of granular material, required for each section.
5. Convert the granular overlay thicknesses to a practical design consisting of an asphaltic concrete surface with or without granular base. In converting thicknesses, each 1 inch of asphaltic concrete surfacing should be considered equal to 2 inches of granular base. When a granular layer is included in the overlay design, its thicknes should not be less than 4 inches.
6. Establish an appropriate grade line to provide the overlay design thicknesses and a satisfactory surface profile.

UTAH

Based on extensive study of the use of pavement deflections to predict the performance of flexible pavements, Utah has developed a procedure for designing overlays by using deflections (19) . It is based on the extension of the AASHO Road Test results and the use of the Dynaflect, i.e., the AASHO equation relating deflections to 18-kip axle loads and remaining life as modified for the Dynaflect is assumed to apply to Utah conditions.

The Dynaflect applied a 1,000 lb. vibratory load at the rate of 8 cps. The load is applied through two 16-inch rigid wheels located 20 inches apart. Five geophones measure the deflections and are located between the load wheels 1 foot apart as shown in Fig. 22. Several deflection basin parameters are used in evaluating the structural condition of the pavement. They are shown in Fig. 23.

By observing the DMD, SCI, and BCI, a qualitative analysis can be made of the pavement structure. Utah has developed a structural diagnosis chart based these three parameters, which is shown in Fig. 24. It is noted that DMD is the maximum deflection, SCI is related to the load carrying capability of the surface course, and BCI shows the relative strength of the lower levels in the structure. In taking field measurements to ascertain the deflection of the existing pavement structure, the frequency of the readings is based on a visual inspection of the roadway. In general, the readings are made at 500-foot intervals. More data are advisable where heavy cracking and wear are apparent. It is noted that Utah assumes fall deflections to be proper for overlay design as measured between June 1 and October 30 and that temperature is not critical within the time band.

When it has been decided that the roadway will be sufficient for the design period and that the pavement and subgrade are structurally adequate, the DMD is used to determine the desired overlay thickness.

Nomographs have been prepared by the Utah Department of Highways utilizing the Dynaflect maximum readings. The DMD 80th percentile is used to calculate the amount of bituminous material necessary to produce a serviceable pavement structure. Also the DMD is used in remaining life analysis to predict the 18-kip equivalent axle loads to failure and to determine the required DMD needed for a projected terminal 18-kip axle loading. The nomographs are shown in Figures 25 and 26.

Once the measured deflection and expected traffic are known, two steps are necessary to determine overlay thickness. First, the required deflection is obtained from Fig. 25, and then the bituminous surface thickness required is obtained from Fig. 26.

Figure 22. Dynaflect layout.

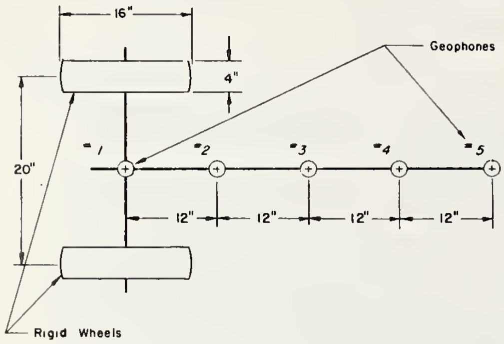


Figure 23. Deflection basin parameters.

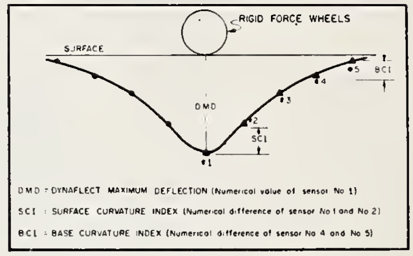


Figure 24. Deflection criteria for Dynaflect.

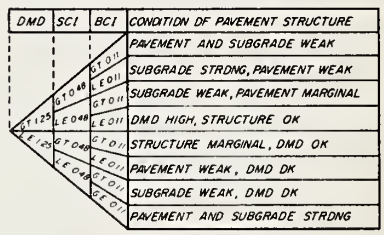
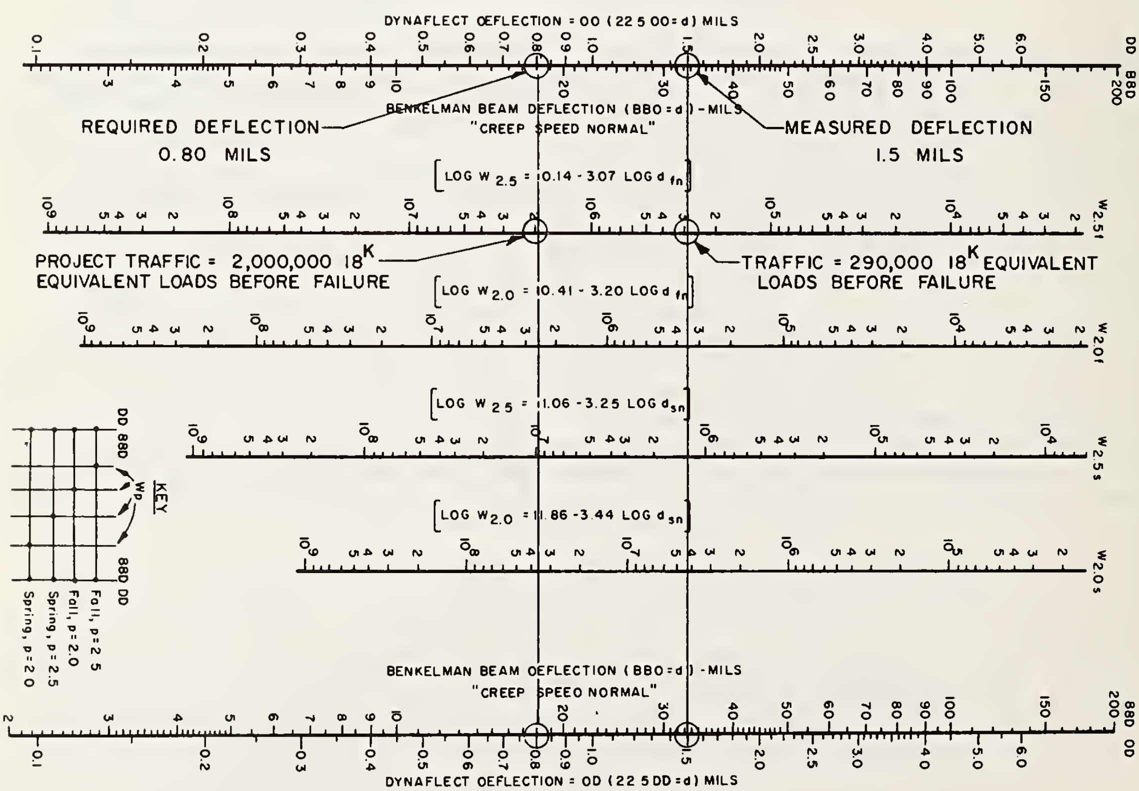


Figure 25. Terminal 18-kip axle loadings and deflections.



The thickness of the required overlay can be calculated, from an equation derived from regression analysis, by knowing the measured deflection and the required deflection. The thickness is determined as follows:

$$T = 5.874+7.288(D_m)-17.828(D_r)-0.800(D_m)+5.038(D_r)^2$$

where

D_m = measured deflection,

D_r = required deflection, and

T = required thickness.

Development of this equation and the nomograph previously presented, is similar to that developed in California, but differences were noted. The graph shown in Fig. 27 was based on the work in Texas using elastic layered theory. Utah cautions that "the design is not advisable for use by other states unless a complete deflection analysis for structural characteristics are investigated."

PENNSYLVANIA PROCEDURE

The Pennsylvania overlay design procedure (20) follows closely that of Utah. An additional aspect of their approach is to more clearly define the effect of temperature changes in the pavement on the deflection measurement. Realizing that the variation of temperature with deflection is complex, Pennsylvania developed a relationship (Fig. 28) from the results obtained at the AASHO Road Test and the research conducted by Southgate and Deen (21).

Upon selecting pavement from serviceability ratings, Pennsylvania uses the Dynaflect or Road Rater for road measurements and determines the characteristic deflection (Dc) for each pavement section. This is simply the sum of the mean deflection of sensor No. 1 and 1.65 times the standard deviation. Eleven readings at 100 feet are taken in representative 1,000 feet per inch. This concept results in a deflection level that is exceeded only in 5 percent of the area of the pavement section. The product of the characteristic deflection and the deflection adjustment factor, which is read from Fig. 28, is defined as the modified characteristic deflection (Dc*). This parameter refers to the pavement deflection at a standard temperature.

The next step in the procedure is to establish required or permissible deflection (Dp) levels. Because setting up permissible deflections for pavement sections has been a subject of considerable controversy, Pennsylvania has chosen to rely on the performance equation of the AASHO Road Test. This equation was evaluated through Benkelman beam measurements, monitored in spring, for a temperature serviceability of 2.5 and was modified to suit that of the Dynaflect. A nomogram was computed and is shown in Fig. 29. Hence, once the number of 18-kip equivalent axle loads the pavement should be designed for is decided, the permissible deflection can be obtained from this nomogram.

The last step in the Pennsylvania procedure is to determine the thickness of overlay required to reduce the measured deflection to the estimated permissible deflection level. This step is accomplished by using a relationship developed by Ruiz (22). For the overlay thickness,

$$h = \frac{R}{0.434} \times \log \frac{D\ \text{actual}}{D\ \text{permissible}}$$

where R is a deflection reduction factor appropriate to the material used in the overlay. This equation was converted to appropriate units, and curves were plotted for various values Dc* and Dp, as shown in Fig. 30. Hence, once the modified characteristics and permissible deflections are established, the overlay thickness for the design life of the pavement section is determined. The Road Rater is currently being used by Pennsylvania and the procedure adjusted accordingly. A computer program has been developed to determine overlay thickness inasmuch as the thickness of structural overlay designed solely on the basis of deflection measurements varies with the number of 18-kip equivalent axle loads. The Pennsylvania procedure cautions the engineer on the extreme importance of applying sound engineering principles in estimating the design 18-kip equivalent axle load applications for the overlay thickness determination.

For successful rehabilitation of a pavement, they also point out several practical steps to be considered in conjunction with the design. Some of them are listed below:

1. The project file of the test section has to be studied carefully, especially variations in the structural section of the pavement, traffic volume and whether revisions are needed, foundation and drainage conditions, and unusual conditions during the construction of the pavement that the engineer feels are likely to affect deflections (e. g., if the asphalt used during the construction of the existing pavement was too hard, surface distress will result although the structural performance of the pavement will still be sound). Examination of such characteristics can frequently shed light on the reasons for early distress and open the possibility of rectifying the same by a correct and economical approach.

Figure 26. Bituminous surface thickness from deflections.

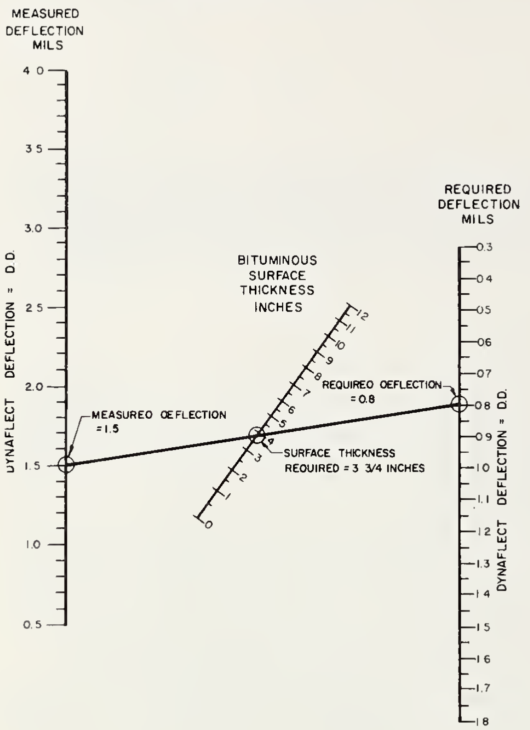


Figure 27. Reduction in deflection.

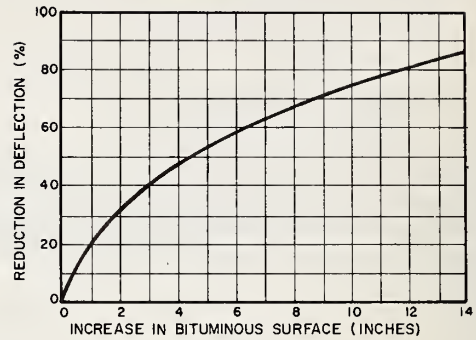


Figure 28. Temperature versus deflection adjustment factor.

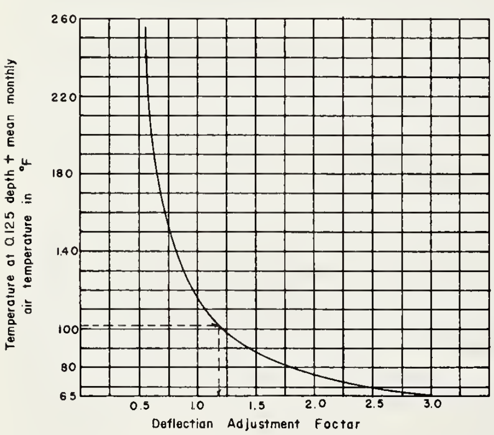
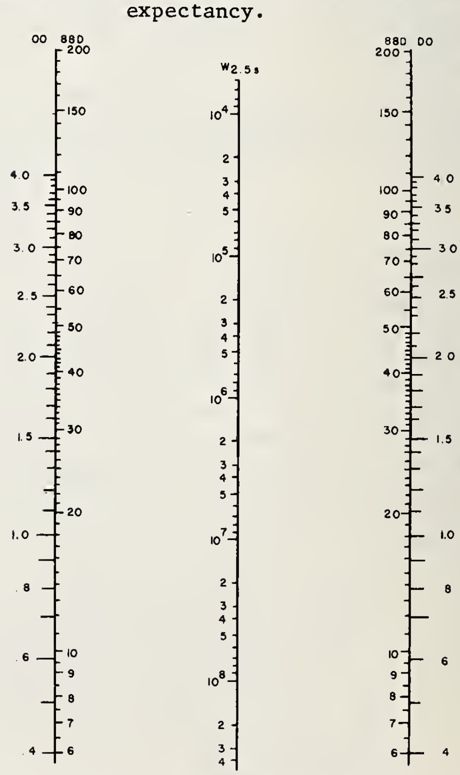


Figure 29. Deflection versus life expectancy.



$W_{2.5S}$ – Number of 18kip axle loads before serviceability level of pavement drops to 2.5 for spring deflections.

2. Isolated areas of advanced distress should always be subjected to substantial dig-out repair prior to the application of corrective treatments.

TRANSPORT AND ROAD RESEARCH LABORATORY

The long-term performance of flexible pavements, as determined by deflection, has been studied in Great Britain over the past 15 years. This work has been carried out on full-scale road tests and on experimental sections, and an overlay design procedure based on deflections has been established (23-26). It is noted that, because of financial conditions, the concept of anticipating major maintenance expenditures on a road as it deteriorates toward failure by applying an overlay before distress appears is seldom realized. Instead of the input being deflection (from regularly conducted surveys as originally envisioned) the first input is normally the report of major distress that results in a more detailed visual condition survey and a deflection survey.

Deflections are measured in conjunction with a 7,000 lb wheel load moving at creep speed by the use of a Benkelman beam. The Lacroix deflectograph, designed to make rapid automatic measurements of deflection and capable of surveying up to 12 miles of road per day with readings at intervals of 11 feet, has been modified to suit British conditions and has been operational for 2 years. The operation of the deflectograph is shown in Fig. 31. The use of the deflectograph as a survey tool has advantages and disadvantages. It gives too close a coverage of results and is inflexible in that the spacing of measurements is fixed. Furthermore, the geometric configuration of the measuring system on the ground is such that measured deflections are considerably less than real deflections on stiff pavements, more so than in any of the modes of operation of the Benkelman beam.

It is also noted that the deflection method using the deflectograph does not apply to extremely weak or strong subgrade conditions (CBR 2 or CBR 10). If applied to weak subgrades the recommendation would probably be conservative, whereas in strong subgrades the opposite is true. The deflection output from the deflectograph is obtained both in a digitized printout and in analogy form, the latter indicating satisfactory functioning of the machine.

If an input of major pavement distress comes though in summer or winter when deflection is less accurate in predicting the required overlay, primarily because of errors in correcting for temperature, a survey is nevertheless normally carried out to give a first indication of rehabilitation requirements. It is then repeated, if time permits, in the following spring or autumn to give greater precision.

The deflection measurements preferred are those made in spring when temperatures most suitable for deflection measurements coincide with high water table and therefore weak subgrade condition. They are corrected for the effects of temperature to standard deflection values, these being the deflection equivalent to a surfacing temperature of 20 C. Fig. 32 is typical of data that have been used to prepare charts for the correction of deflection measurements to standard values. It is noted that use of this chart may lead to inaccurate estimation of standard deflection when pavements, known to be sited on either very strong or weak foundations, are tested at temperatures close to the extremes of the normal working range.

In establishing the deflection history of the pavement in the immediate area of measurement, the pavement is classified as being sound, critical, or failed. Development of critical conditions indicates the need for pavement strengthening to avert structural failure. The classifications are associated primarily with rutting in the wheel paths and cracking. A description of the classification is given in Fig. 33.

Analysis of the relation between deflection and performance has been made in terms of standard early life deflection (SELD) values normally taken as the mean spring deflections over 3 or 4 year period. Because almost all pavements show a slow increase of deflection with time, the significance of a given level of deflection in predicting future performance depends on the age of the road when the deflection is made. The relation between deflection and traffic up to the onset of critical pavement conditions is shown in Fig. 34 depicting rolled asphalt bases. For pavements with crushed stone and coated macadam bases under rolled asphalt surfacings the curves are similar. Acceptable deflection levels are lower on pavements with cemented bases. Also shown are mean trend lines of deflection from early life of the pavement from this diagram, once the existing deflection and the past cumulative traffic are known.

Utilizing the deflection criterion curve, one is able to determine the design deflection level for the required design life of the pavement. The final step is to determine the thickness of the overlay needed to reduce the mean deflection plus one standard deviation of each section to the design level. This overlay thickness is determined on the basis of experience. It has been concluded that for pavement with subgrades of weak to moderate strength (CBR 2.5 to 10), the reduction in deflection is largely independent of subgrade stiffness and the type and thickness of the overlayed material. Recommendations by the British are given in Fig. 35. Pavements with granular and bituminous bases that are close to or in the critical condition are given in Fig. 36. It is considered that these curves may be slightly conservative, but, in light of the factors that

Figure 30. Overlay required based on Dynaflect deflections.

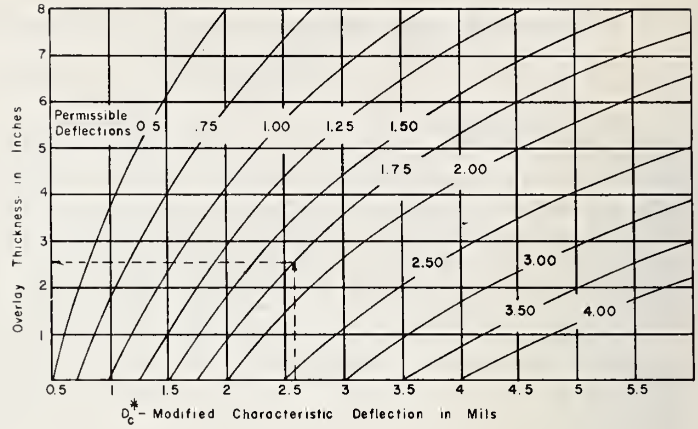


Figure 31. Diagrammatic representation of Deflectograph.

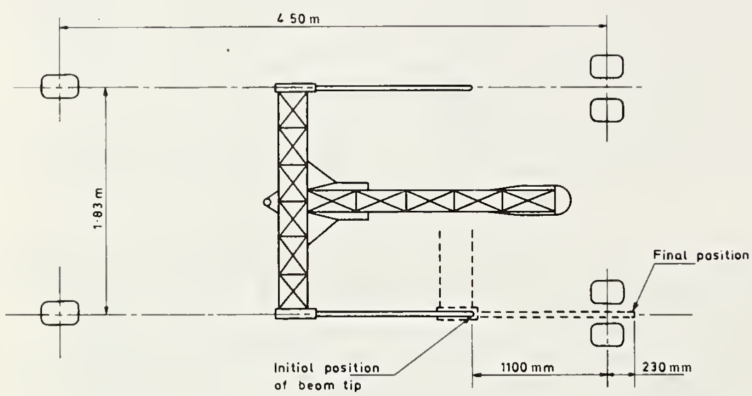


Figure 32. Chart for estimation of standard deflection.

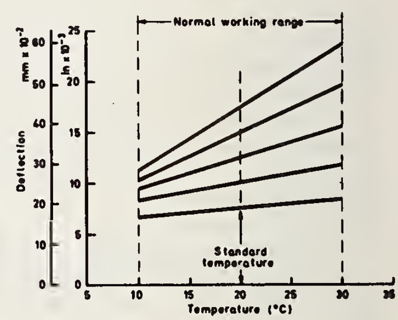


Figure 33. Classification of pavement conditions.

Classification		Visible Evidence
Sound	1	No Cracking. No appreciable rutting
	2	No Cracking. Rutting less than 10 mm
Critical	1	No Cracking. Rutting between 10 mm and 20 mm
	2	Cracking confined to single crack in wheel path. Rutting less than 20 mm.
Failed	1	Cracking extending over the area of the wheel path and/or rutting greater than 20 mm.
	2	Dangerous to traffic. Disintegration of the road surface.

Figure 34. Relation between deflection and traffic up to onset of critical pavement conditions.

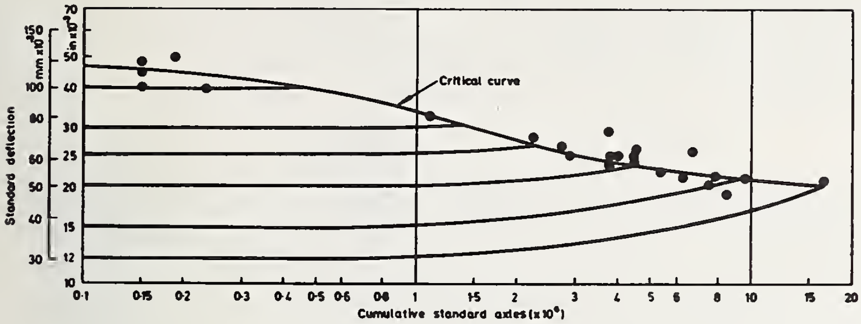


Figure 35. Reduction in deflection achieved by overlays of different thicknesses.

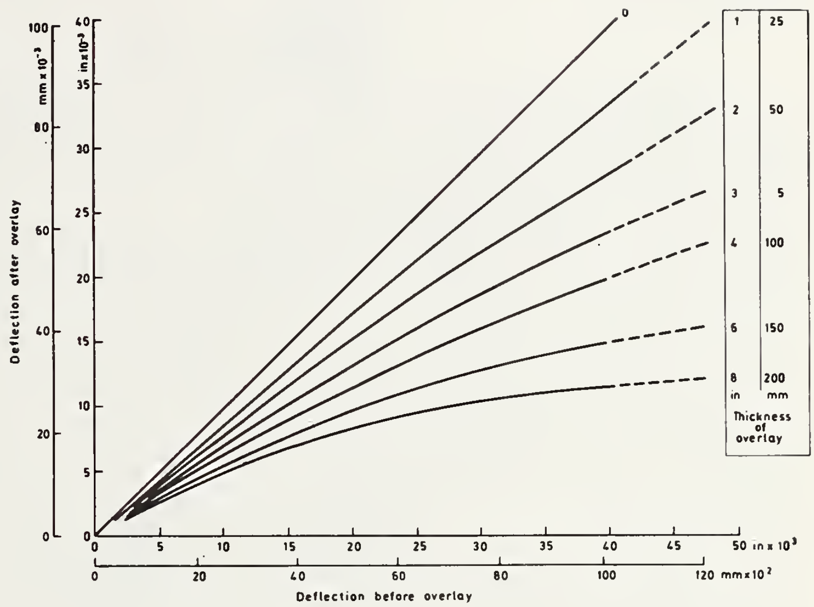
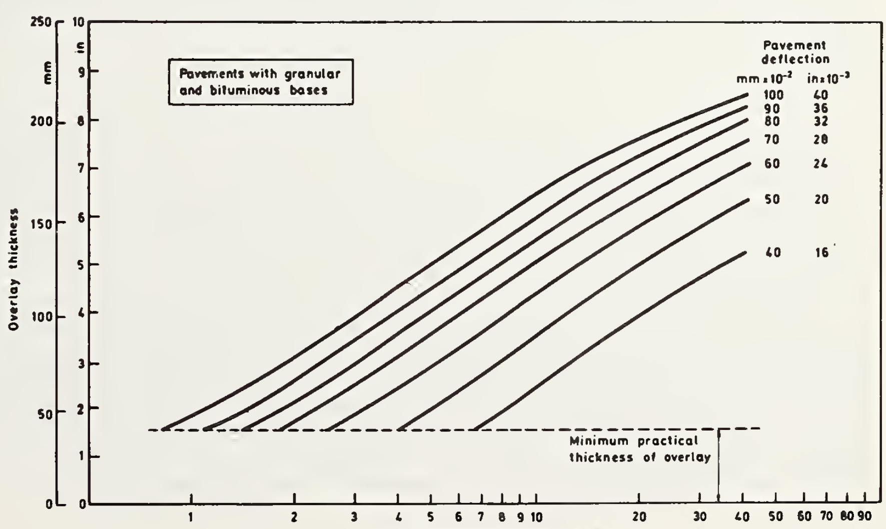


Figure 36. Thickness of overlay required to achieve a given extension of pavement life.



may affect overlay performance, the approach is considered to be realistic. It is noted that the results of a deflection survey for overlays divide the pavement into lengths over which the deflection levels are reasonably constant. This is to avoid frequent variations of overlay thickness. In practice changes are not normally made at intervals of less than 440 yards. On each length, an overlay of constant thickness is then designed to reduce the deflection level to achieve the required extension of life over most of the length (normally 95 percent). Isolated areas of very high deflection may require reconstruction and are dealt with separately.

TEXAS METHOD

As asphaltic concrete overlay design subsystem for the Texas Highway Department has been proposed by Brown and Orellara (27). This subsystem provides several AC overlay strategies for existing flexible pavements, which are listed in order of ascending cost. Dynaflect deflections are used to characterize the existing pavement structures and the AC overlay material. The flow chart (Fig. 37) summarizes the procedure used in the Texas computer program (overlay 1). The procedure is based on the deflection-performance equation developed by Scrivner for pavement design in Texas. Uncertainty in the predicted pavement life, as a result of the variability in the materials characterized, is treated so that the designer may select either a 95 or 99 percent confidence level.

OKLAHOMA METHOD

The Oklahoma overlay design procedure is based on providing a psi of 2.0 after 20 years of service (28, 29). From the flow chart in Fig. 37, it is seen that this overlay design method entails three separate subsystems: a pavement condition survey, deflection measurements, and a flexible pavement design procedure. Each is considered independently without assigning weighting factors to the thicknesses derived from each.

For the pavement condition subsystem, data obtained from a detailed pavement condition survey are used to assign a pavement rating from 0 to 100 percent (failed to excellent). The depreciation (100 minus the rating), the pavement age--counting from the last major rehabilitation or new construction (Fig. 39)--and the intersection gives the expected design life. The expected design life curve is then used to determine the deterioration expected 10 years hence. This information is then used with Fig. 40 to estimate the Benkelman beam deflection. The required overlay thickness is then determined by means of Fig. 41.

The second method consists of measuring the pavement deflection with a 9,000 lb wheel load. The deflections are averaged for areas of similar deflection, and the average is adjusted for seasonal variation. The corrected deflection value is used to determine the required overlay thickness of AC from the nomograph in Fig. 41. The amount of overlay is a function of the average deflection greater than 0.022 inch. This limiting value was derived from a field study of in-service pavements, which found the maximum allowable pavement deflection with a 9,000 lb wheel load should not exceed 0.037 inch for a pavement to perform satisfactorily for 20 years. A safety factor of 33 percent and a fatigue factor of 33 percent were used to convert to a 15,000 lb design wheel load, which is equivalent to a maximum deflection value of 0.022 inch. This conversion was made because the design wheel load for Interstate highways is 15,000 pounds.

Flexible pavement design procedure is a reevaluation of the design using the Oklahoma Department of Highways flexible pavement design method. The existing pavement structure is subtracted from the pavement structure--derived by reevaluation--to arrive at the overlay thickness required.

VIRGINIA METHOD

The Virginia overlay design procedure is used for resurfacing flexible pavements on primary, Interstate, and arterial roads.

Extensive studies in Virginia have shown the pavement deflections predict the performance of pavements. Based on the deflection studies, an overlay design procedure has been developed that uses maximum deflection and the shape of a deflected basin for determining the thickness of an AC overlay.

Pavement deflections are determined by Dynaflect as shown in Fig. 38. The Dynaflect deflection d_1 on the No. 1 geophone (Fig. 38) at the center of the two applied loads is converted to a maximum deflection d_{max} of an 18-kip axle load by an equation $d_{max} = 28.5 \times d_1$. As a means of interpreting the shape of the deflected basin, a factor termed spreadability is used. This factor is the percentage ratio of the average of the five deflections--under geophones 1 through 5--to the maximum deflection measured at the center of the two applied loads. An increase in this factor indicates an ability of the pavement to spread the load over a wider area. Thus a spreadability of 60 indicates a stiffer pavement than does one of 40.

Figure 37. Summer flow chart for program overlay.

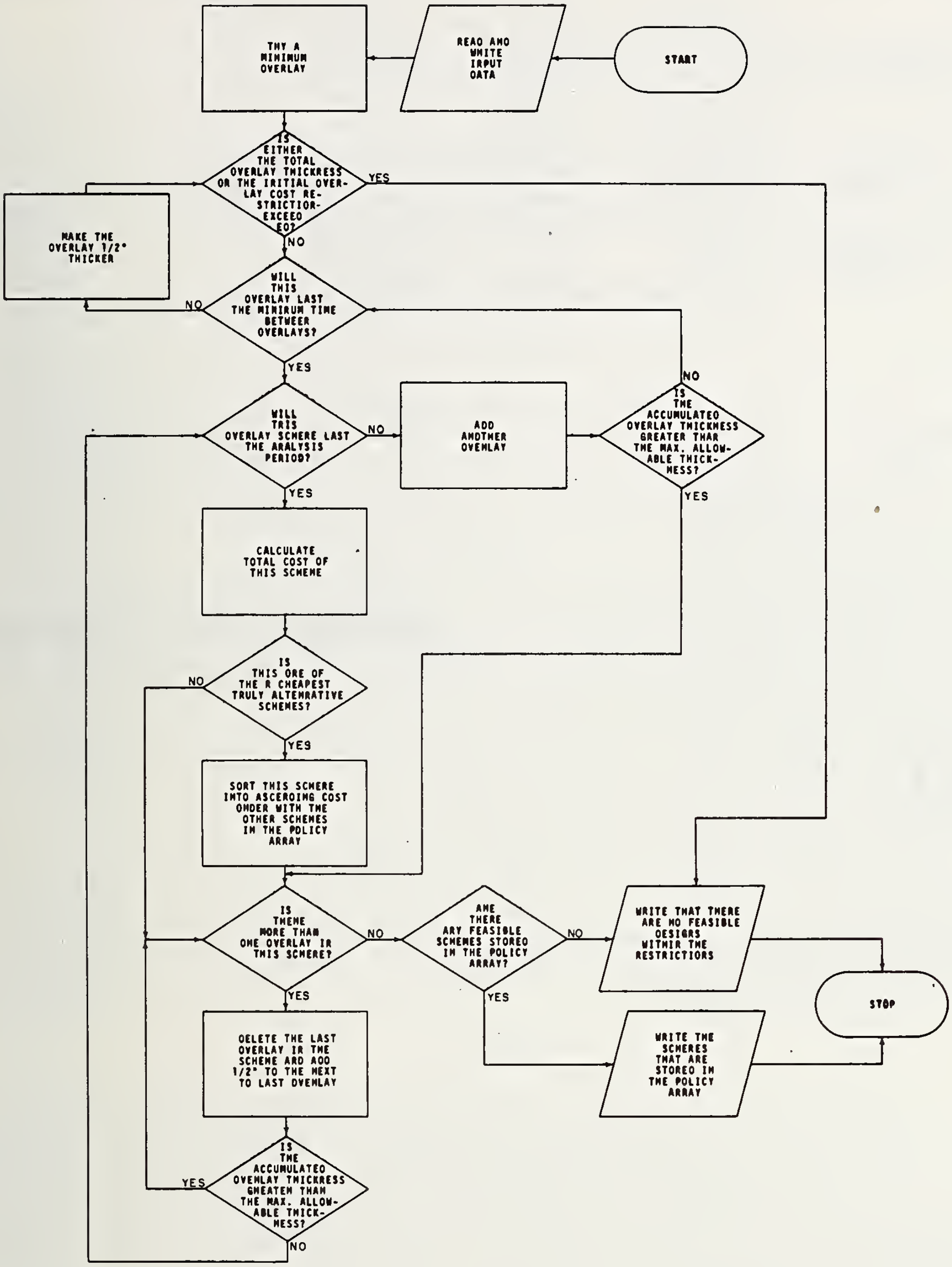


Figure 38. Overlay design procedure for flexible pavements.

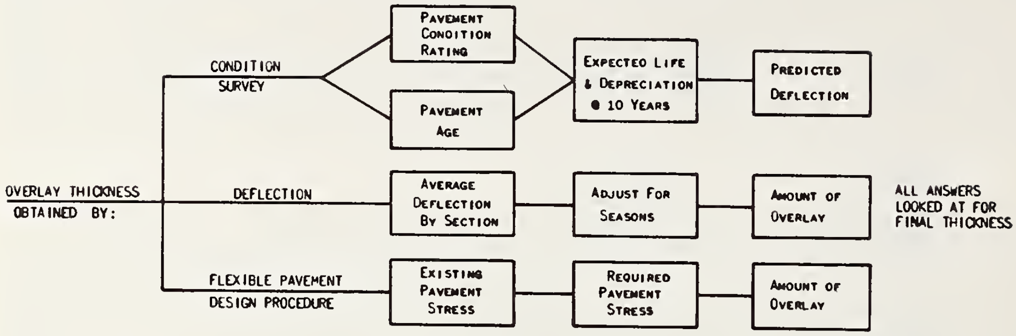


Figure 39. Life curves for flexible pavements.

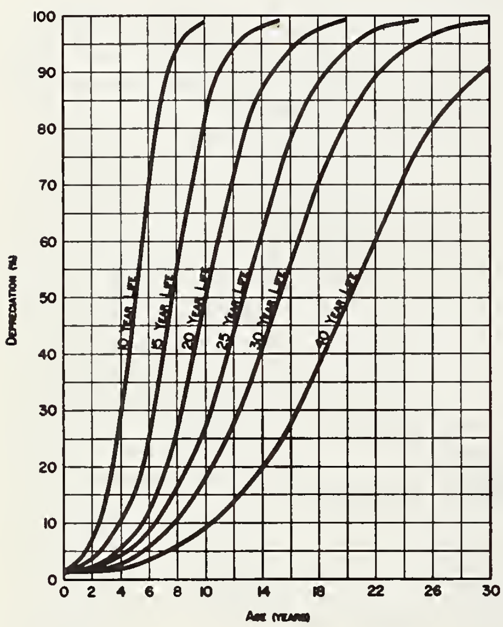


Figure 40. Deflection versus depreciation for flexible pavements.

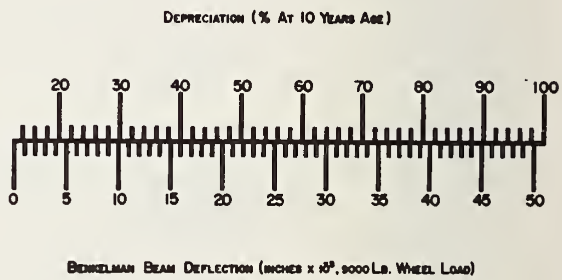
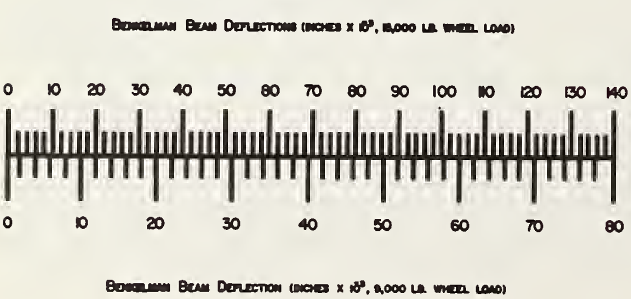


Figure 41. Overlay requirements based on deflections.



The design is based on the following:

1. AASHO Road Test result model equation $D = a_1 h_1 + a_2 h_2 + \ldots$ where D = pavement strength, a = the thickness equivalency of AC and is considered as equal to 1, and h = equivalent thickness of AC in inches. a_1 and a_2 are thickness equivalencies of the materials in the layers of the pavement having thicknesses h_1 and h_2 respectively.
2. Burmister's two-layer theory with the pavement as the top layer overlying the subgrade layer of semi-infinite depth. Because the top layer consists of a number of layers of varying thicknesses and varying strength moduli, the pavement is assumed to have an equivalent thickness h with a strength modulus equivalent to that of asphalt concrete.

Based on the model equation from the AASHO Road Test results and the two-layer theory, an overlay design chart (Fig. 42) has been prepared that correlates the following variables:

1. E_s = the subgrade modulus in psi,
2. h = the equivalent thickness of asphalt concrete in inches,
3. d_{max} = maximum rebound deflection under an 18-kip axle load in inches, and
4. S = spreadability of the deflected basin.

Thus as an example, given a maximum rebound deflection d_{max} = 0.05 inch and a spreadability S = 60 the design chart shows that the pavement with such deflection data has an equivalent thickness h = 5.9 inches of plant mix and a subgrade modulus of 3,600 psi.

From the Virginia pavement design chart (not given here), if it is determined that for a given soil support value and design traffic (in terms of 18-kip equivalent) the thickness of asphalt concrete (full depth) needed is more than 6 inches, an overlay is justified to obtain the required structural strength of the pavement. If the thickness of full-depth asphalt concrete needed is 8 inches, a 2-inch thickness of premix overlay is required.

The advantages of this method are as follows:

1. It is a rational basis of design based on AASHO Road Test results successfully applied in Virginia for the last 5 years for design of primary, Interstate, and arterial roads. The method also incorporates Burmister's two-layer elastic theory, which has been found to give a reasonably correct approximation in pavement design and evaluation.
2. Investigations in Virginia have shown that, where very high pavement deflections are obtained because of weak subgrades, an overlay becomes less effective and may, in extreme cases, be inadvisable. By this overlay design method the strength of the existing subgrade is also obtained, which will determine the amount of additional strength the pavement would achieve after the overlay is provided. In the example mentioned above if the equivalent thickness of the pavement is increased from 6 to 8 inches by providing a 2-inch overlay, the deflection d_{max} of the modified pavement should reduce to 0.038 inch as shown in the Fig. 42. This may not happen because of low subgrade values (3,600 psi), and the deflections obtained would be higher than 0.038 inch resulting in an effective pavement thickness of less than 8 inches.

This overlay design method continues to remain under study, though the results obtained from the few studies carried out to date have been very encouraging.

Because the overlay design is based on the design for a new pavement, the modified pavement, after the overlay is provided, could be considered as providing a life equivalent to the life of a new pavement.

OBSERVATIONS

Several interesting observations can be made concerning pavement life and the structural performance of a roadway in relation to deflection measurements. These relationships are the underlying basis in the development of several of the flexible pavement overlay procedures previously discussed: (a) for adequately designed pavements, deflection taken during the same season each year remains fairly constant for the life of the pavement, (b) there is a tolerable level of deflection as a function of traffic that can be established based on the fatigue life of the pavement, and (c) overlaying a pavement will reduce its deflection and thickness needed to reduce it to a tolerable level can be established.

A typical deflection history curve, representing the performance of a well design pavement structure, is shown in Fig. 43. The strength of the pavement (as reflected in deflection measurements) undergoes three phases in its behavior:

1. The initial phase, immediately following construction, in which the pavement structure is further consolidated and strengthened and the deflection shows a slight decrease.
2. The functional phase during which the pavement will carry the anticipated traffic without undue deformation and the deflection will remain fairly constant or may increase slightly.
3. The failure phase occurring under the repeated stresses of both traffic and climate where the deflections greatly increase. This phase results in rapid deterioration and pavement failure. Fig. 44 shows a typical structural performance curve for a pavement structure

Figure 42. Overlay design chart.

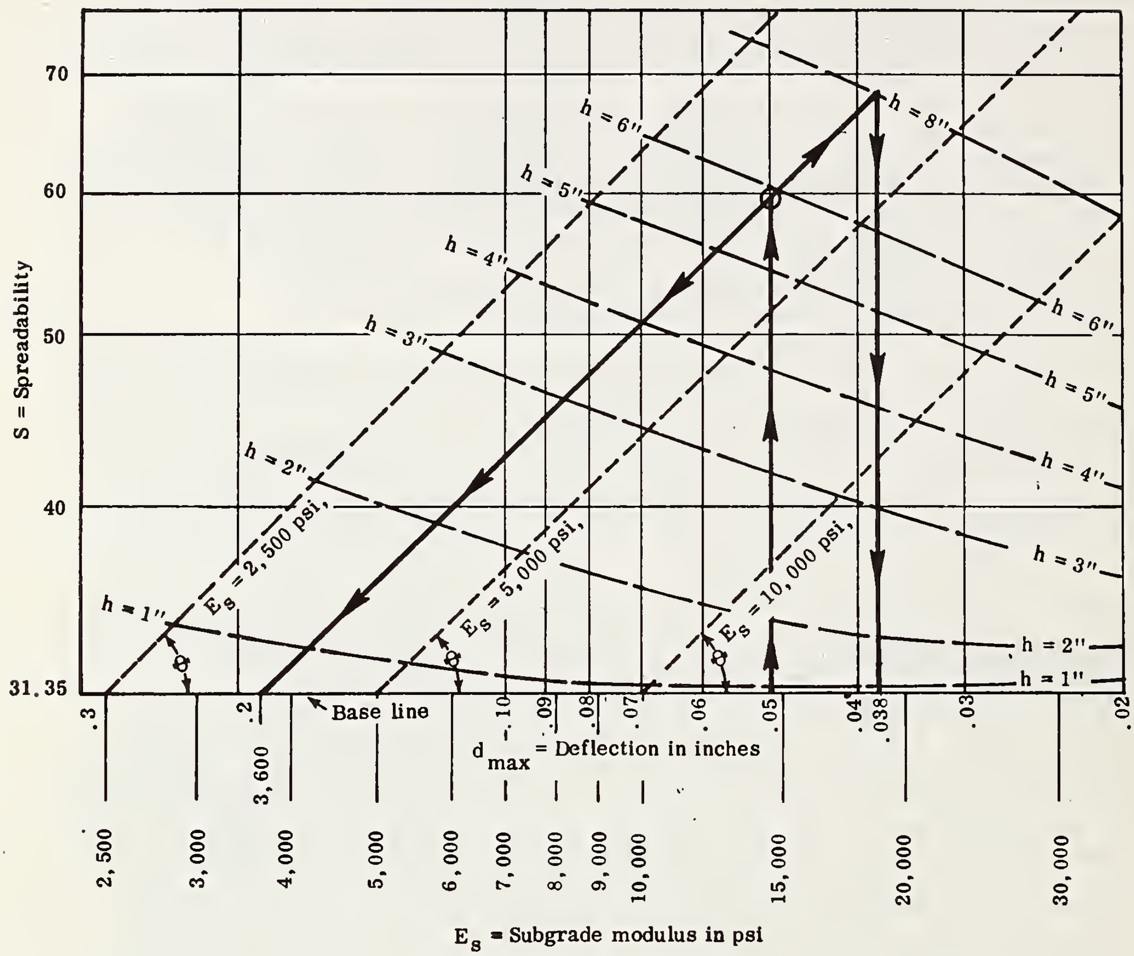


Figure 43. Well-designed deflection-history curve.

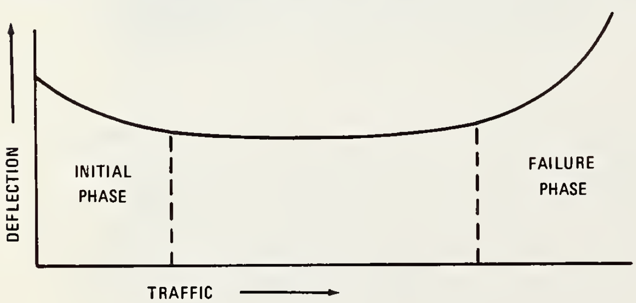
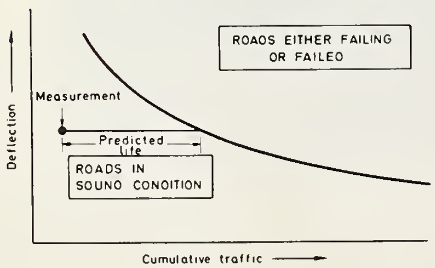


Figure 44. Underdesigned deflection-history curve.

Figure 45. Deflection criteria curve.



that was underdesigned initially.

Deterioration is quite rapid and may be caused by weakness in the foundation materials or in the pavement structure itself.

In general, any design procedure that has a design criterion based on traffic can be worked backward and some remaining life analysis can be made. With the concept of a tolerable level of deflection, an analysis as shown in Fig. 45 can be made to give a rough estimate of the remaining life. The measured deflection value is plotted at the existing traffic. By carrying this deflection value horizontally to the tolerable curve, the remaining life can be estimated. Measured deflections above the tolerable curve indicate that immediate rehabilitation is necessary. This type of remaining life analysis is incorporated into the Asphalt Institute and Utah procedures. Although the California method has no specific requirement for remaining life, this analysis can be used. As is readily apparent, most overlay design procedures are based on experience with deflection measurements or with a modification of an existing pavement design method. Limited work has been done, on a theoretical basis, to reinforce these approaches.

There is however considerable interest in this area. Work at TTI (40) has produced a method to determine values of elastic moduli for a two-layer system from Dynaflect readings Work is under way to expand this to a three-layer system. At Ohio State, Majidzadeh is pursuing a similar approach with the Dynaflect. He is also using it in subgrade compaction evaluation. Similar work has been done by Vaswami in Virginia.

Attempts to utilize wave propogation techniques for highway pavement evaluation have been very limited and somewhat disappointing. They have been more successful on airfield pavements (41).

SUMMARY

A wide variety of procedures developed by many agencies do exist, and experience is a major factor in most procedures. For flexible pavement deflection-based procedures dominate, whereas for rigid pavements modification of normal design thickness procedures based on Corps of Engineers or other tests are used to a large extent. Use of new materials (fibrous concrete) and new concepts (elastic jointed and thinner concrete overlays) are gaining national attention. Additional work is expected in these areas.

It is expected that future emphasis will be on reconstruction and rehabilitation of our existing highways within present rights-of-way. The topics of pavement rehabilitation, more specifically that of overlay design determination, are of immediate concern to the highway engineer and will remain so over the next several years.

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STRUCTURAL EVALUATION AND OVERLAY DESIGN METHODOLOGY FOR AIRFIELD PAVEMENTS: STATE OF THE ART

M. W. Witczak

This report is a state-of-the-art review dealing with the structural evaluation and overlay design procedures for airfield pavements of flexible, semi-rigid, and rigid construction. In airfield pavement terminology, evaluation encompasses evaluation of an existing pavement as to its surface condition from both an engineering viewpoint (e.g., skid resistance) and a functional viewpoint (aircraft response to surface irregularities), evaluation of the load carrying capacity, and evaluation of existing pavement life (16). Each of these considerations forms an integral part of the overall assessment of an airfield pavement evaluation analysis.

The structural evaluation procedures discussed in this report are confined to the manner in which either the component layer material properties or total pavement system response are evaluated for use as input into a structural analysis to determine the overlay thickness requirements of an airfield pavement.

HIGHWAY AND AIRFIELD PAVEMENT DIFFERENCES

To fully appreciate the present state of the art in airfield pavement overlay analysis relative to that of highways, it is important to recognize basic differences in design approach and philosophy.

The most obvious difference occurs in the magnitudes of gross weights encountered. In highways, this factor does not change with time because of restrictions imposed by various authorities on legal axle load limits. In contrast, airfield analysis reveals a marked gross load increase over the past 25 years as shown in Figure 1. In designing a new pavement or overlaying existing surfaces, the design needs frequently dictate the selection of a heavier aircraft type commensurate with projected utilization estimates. In rare instances, when new major intercontinental airfields are designed, the designer must use extrapolations of Figure 1 concepts to arrive at a design for critical aircraft of 1,000 to 2,000 kips. In addition, scant input relative to the gear type, number of tires, gear arrangement, and tire pressures is forthcoming. Thus in some cases a critical aircraft may be selected that is not presently in a production stage by aircraft manufacturers.

Another important feature illustrated by Figure 1 is the need by airport operators to update pavements to accommodate the continual changes in aircraft size and weight. Many airfield pavements are frequently overlaid in order to increase the load capacity of an existing surface that is performing well and showing few signs of distress.

The importance of this feature may be better explained in terms of the remaining damage D_R of a pavement structure. It is a logical conclusion from cumulative damage theory that if an overlay is placed on a failed pavement, the remaining damage of this existing pavement must be zero (D_R=0). There is no justification to resort to this theory for overlay design purposes. However, if an overlay is placed primarily for strengthening (increased load capacity and/or pavement life), $D_R > 0$, and the applicability of this theory is quite apparent.

Because of the heavy loads involved in airfield overlay analysis, the deep stress zones of influence created by the aircraft in pavement substructure must be considered. As an example of the magnitude of this problem, a proposed felxible pavement structure for the new Honolulu Reef runway is shown in Figure 2. A multilayered theory analysis of this structure for a 2,000-kip design aircraft load established that the controlling layer (from a shear deformation viewpoint) was the ocean bottom located 14 feet below the pavement surface.

Though this example is representative of an extreme design condition, it calls into question the use and applicability in airfield work of light, nondestructive, vibratory testing devices that are currently considered adequate for highway analysis. Finally, it should be realized that, in addition to shear deformation, density requirements for the new critical aircraft must be met at all depths to ensure against deformations due to densification.

Heavily trafficked highway pavements frequently are designed for 1,000,000 or more repetitions of a standard 18-kip axle load. In contrast, most airfield design procedures recognize that 5,000 to 25,000 (maximum) strain repetitions would be equivalent to capacity operation in a 10 to 20 year design life. While this is not the same viewpoint shared by the author, it nonetheless forms the basis of many recognized airfield design procedures currently in use. This concept is directly attributable to why the use of a single design or critical aircraft is selected as the primary load and traffic parameter in nearly all current airfield design procedures. For most methods, the factor of traffic repetition is considered indirectly as capacity operations are assumed. This is in contrast to many highway design procedures that have the flexibility to directly account for any value of design repetition for the pavement life.

It can be inferred from the preceding discussion that there is no need to have a methodology for equating the damage effects of mixed aircraft traffic. This again contrasts the accepted procedure of using equivalent damage factors for various highway wheel loads in the design analysis. Only recently have design procedures been advanced to compensate for the effects of mixed aircraft traffic on pavement behavior. Some examples are the 1970 change 2 to the FAA Airport Paving Manual (39); the 1973 edition of the PCA design manual (104), which contains an alternate procedure for checking design thickness against accumulated fatigue damage for the anticipated design mix; and the 1973 full-depth asphalt airfield design procedure of The Asphalt Institute (127). The latter procedure strictly relies on the concept of equivalent aircraft damage factors in lieu of a critical design aircraft.

Despite the fact that a procedure for airfield analysis using equivalent damage concepts for mixed aircrafts effects possesses many advantages over a procedure relying on a critical aircraft, a need exists for airport operators to have an implementable system for evaluating the load carrying capacity of all pavement systems.

One final but important consideration is the fact that an airfield system has several distinctly different pavement areas (aprons, taxiways, and runways). Each has a very specific and individual function and a different set of failure criteria. In addition each area must be evaluated in terms of the variable degree of safety that must be afforded in the design consideration. On a highway, a driver has the option to adjust the speed of his vehicle to take into account the present condition of the highway. However, this option is not available to an aircraft pilot during takeoff and landing when certain critical velocities must be attained.

In summary, many significant differences between highway and airfield design concepts should be kept in mind. In airfield analysis, extremely heavy gear loads are generally encountered. In addition, the magnitude of the design aircraft is continually changing. This leads to the concept of overlays for strengthening existing pavements more frequently than for complete rehabilitation of a failed structure.

Traffic repetitions are indirectly involved in most current design procedures by assuming capacity traffic levels during the design life, using thickness adjustments for various airfield traffic areas, and using variable safety factors for material strengths (variable working stress levels) associated with various pavement areas.

As a result, structural design is based primarily on a critical aircraft type rather than repetitions of a standard aircraft expected in the design life. A direct result is the general absence of analysis using equivalent load (damage) factors for mixed aircraft effects. This fact presents an obvious difficulty that must be circumvented if remaining damage concepts are to be successfully employed in airfield overlay analysis.

The presence of heavy gear loads in airfields implies that deep pavement layers may be of extreme importance, especially in view of the possibility of using light non-destructive methods of material evaluation. Finally, failure criteria differ within airfield pavement areas and would logically appear to vary between highway and airfield design methodology.

CURRENT AGENCY DESIGN OVERLAY PROCEDURES

This section deals with a summary of the most salient features of the overlay design procedures and methods set forth by various agencies. Effort has been made to subdivide the FAA procedure into concepts presented in the current FAA manual and procedures stated in the tentative proposed change 4 to the FAA manual. (It should be realized that discussions of FAA proposed change 4 are not to be construed as changes that will definetely be incorporated into AC 150/5320-6A. The changes noted are to be considered as provisional in nature and are only stated to give additional insight into what possible changes may occur.)

Every overlay design procedure shown requires the solution of a new pavement design as an integral part of the overall overlay procedure. This is true for both rigid and non-rigid designs. For purposes of brevity, the intricacies of each new design procedure are not stated in this report inasmuch as each method is well documented in the literature.

In addition, many details of design not related to the structural conditions must be considered for a proper overlay analysis. Problems such as jointing arrangements in rigid over rigid overlays, load transfer, frost requirements, and subbase thicknesses in rigid pavements are all of utmost importance. However, these features are considered to be outside the major purpose of this study, and again the details regarding these factors may be found in the appropriate references.

General Overlay Categories

Unlike highway analysis, the primary factor involved in the specific overlay procedure used in airfield analysis is dependent on the type or category of overlay to be used. Based on an examination of the various types considered by all the agencies investigated, 9 specific categories have been found. These are illustrated in Figure 3. The overlay classification adopted for this report is based on (a) the type of overlay used, (b) the type of existing pavement, and (c) the "behavior" of the pavement system.

Overlay types are normally classed as being rigid (plain or wire reinforced concrete) or non-rigid. The latter may be of a flexible (sandwich) variety if unbound granular material is used in the overlay or bituminous if the entire non-rigid overlay thickness is composed of a bituminous bound material.

Existing pavement types are classified as either being one of three categories: rigid, flexible, or composite. The third consideration, that of defining pavement system behavior, is concerned with the evaluation condition of the existing pavement and how the overlay reacts to the load. For example, if an existing rigid pavement is badly distressed, its behavior may approach that of a flexible system in its ability to transfer the load by slab action. In another situation, even if an existing rigid pavement is uncracked, a non-rigid overlay may be of such thickness that the pavement response to the load is primarily due to the non-rigid overlay, and as such the rigid effect from the existing slab is outside the major zone of the pavement response.

Non-Rigid Over Flexible - This overlay category is typified by either a flexible or bituminous overlay placed on an existing flexible pavement. The overlay is adaptable to strengthening either a failed or unfailed pavement system. Generally there are minimum thicknesses for both the asphalt concrete and unbound granular layer. These two specifications when combined normally make it feasible to use total bituminous overlays for thicknesses of overlay less than 7 to 9 inches depending on the agency and design factors selected. Some agencies, however, do not recommend the use of flexible overlays and use a bituminous overlay regardless of the thickness of overlay.

Non-Rigid Over Composite - This is a flexible or bituminous overlay placed over an existing composite pavement. Normally the composite pavement itself is composed of an existing rigid pavement that had previously been overlaid with a non-rigid overlay. Minimum thickness requirements and use of both asphaltic concrete and unbound granular layers are similar to those discussed previously.

Non-Rigid Over Rigid (Rigid System) - This represents a flexible or bituminous overlay placed on an existing rigid pavement considered to be structurally intact. Application is limited to designs involving strengthening of a rigid pavement rather than complete pavement rehabilitation.

Non-Rigid Over Rigid (Flexible System) - A flexible or bituminous overlay is placed on an existing rigid pavement that is badly cracked and has lost most of its structural integrity. Some agencies also specify this condition to exist if the thickness of the non-rigid overlay is greater than either the existing rigid slab thickness or a certain minimum overlay thickness.

Rigid Over Flexible - A rigid pavement overlay constructed over an existing flexible pavement is normally adaptable for both strengthening and rehabilitation functions.

Rigid Over Rigid (Rigid-Bond) - A bonded overlay is one that is carefully bonded to the existing pavement. The existing pavement must be in good structural condition as its use is primarily for strengthening rather than rehabilitation. Common thickness ranges for this type of overlay vary from 1 inch minimum to about 5 or 6 inches maximum due to economic considerations. Larger bonded overlays have been constructed and have performed satisfactorily. Surface preparation and bonding agent application are extremely important to ensuring good performance. The surface is normally treated with an acid-etch procedure though sand cement grouts or epoxy mixtures have been used as bonding agents to provide a monolithic slab comprised of the existing pavement and overlay.

Rigid Over Rigid (Rigid-Partial Bond) - The most common rigid overlay placed over an existing rigid pavement is termed the partial bond. The new rigid overlay is placed, for strengthening purposes, directly over the existing pavement without a bonding agent. Influence from the existing

Figure 1. Aircraft gross weight trend.

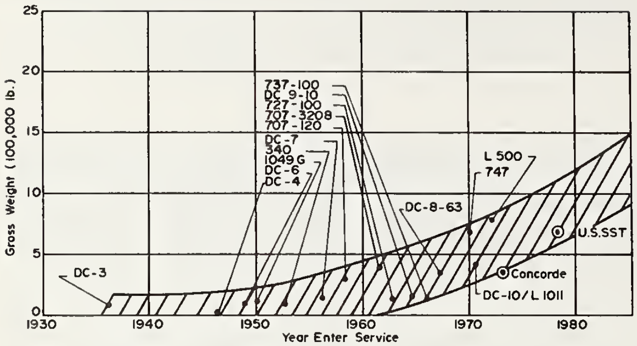


Figure 2. Proposed runway design for Honolulu.

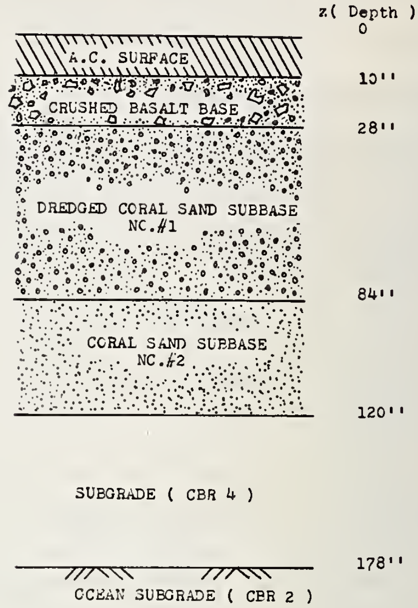
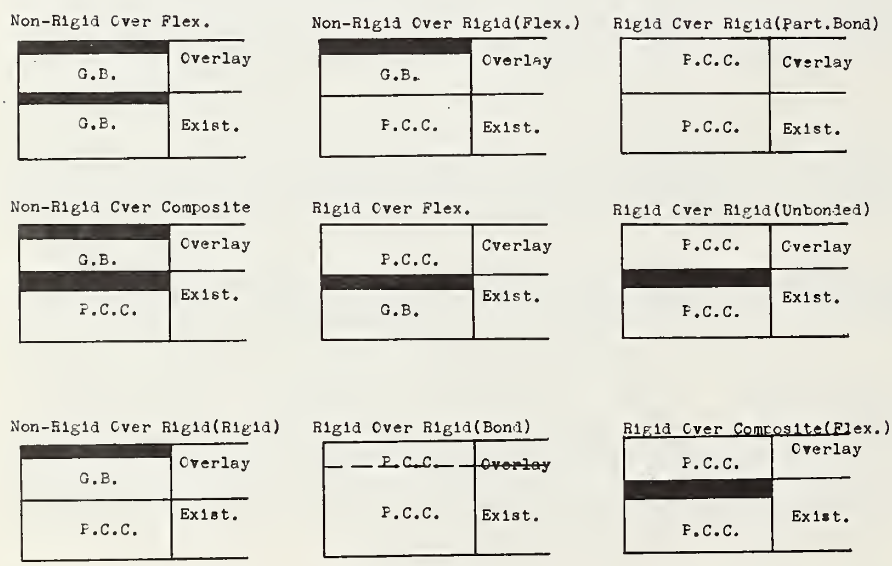


Figure 3. Overlay categories.



slab is accommodated by friction between slabs as well as some bonding. If defects in the existing slab cannot be repaired, this type of overlay is contraindicated.

Rigid Over Rigid (Rigid-Unbonded) - This category represents a rigid pavement overlay placed on an existing rigid or composite pavement structure. However, the thickness of the separating course is thin enough to permit interaction between the existing slab and the new overlay slab. Though the original pavement may be rigid, construction or engineering principles may dictate that a separating course be required to level the existing irregular pavement surface or to increase the grade line. This overlay procedure is applicable for all conditions of the existing pavement.

Rigid Over Composite (Flexible System) - A rigid overlay placed on an existing composite pavement structure, the primary difference between this overlay type and the one above is based on the thickness of the separating course (generally non-rigid) of the existing composite structure. If the thickness of this layer is great enough, the new overlay will act as a new or single slab under no influence from the existing rigid pavement.

Table 1 is a summary of the specific overlay category as recommended in the existing manuals applicable to each agency shown.

Overlay Design Approaches

All of the overlay categories developed from various agencies fall within 2 general design approach philosophies. These two approaches are shown conceptually in Figure 4 and have been termed new design (method 1) and thickness deficiency (method 2).

In the new design philosophy, overlay thickness is based on the concept that the existing pavement serves as the foundation or design subgrade for the overlay thickness. Hence the strength is evaluated at the surface of the existing pavement and the overlay is treated as a new design.

Thickness deficiency (method 2) involves the concept that the design subgrade is the subgrade of the existing pavement structure. Using this as a subgrade support value, a new design thickness is established with the difference in thickness between the existing pavement and the new pavement requirements representing a "thickness deficiency" equivalent to the overlay thickness. In order to obtain overlay thickness, it is imperative that both existing and new thicknesses are expressed in the same equivalent material. Pavement layer condition factors are used for the existing pavement, and material equivalencies may be used to convert the overlay thickness into various material layers.

Overlay Design Equations

Basic overlay design equations and approaches common to each agency and overlay category are shown in Table 2. Despite widespread agency concensus, some variation occurs in the U.S. Navy and TAI methods for non-rigid over rigid overlays and the Canadian Department of Transportation (CDOT) procedure for rigid over rigid overlays. The design equation for the CDOT method is also shown at the bottom of the table. The procedure for the U.S. Navy method is shown in Figure 5. This concept is based on the principle that the non-rigid overlay spreads the load at a 45° angle, thus reducing load intensity as applied to the existing rigid pavement. This procedure is applicable only for the rigid behavior of an existing rigid pavement. For the flexible behavior an alternative method is used based on evaluating the CBR of the existing subgrade and using a layer equivalency value of 2.0 for the PCC pavement (1 inch PCC=2 inch high quality base course).

As can be noted from the table, the only overlay categories where a method 1 approach is used for the overlay thickness are rigid over flexible and rigid over composite-flexible system. All other overlay categories use the thickness deficiency approach.

Structural Evaluation Methods

Several structural evaluation techniques are used by agencies to determine strength properties of the existing layers and/or pavement. All are complemented by standard tests to determine layer thickness, density, moisture etc.

CBR Test - This widely known strength test is used (directly or indirectly) in airfield overlay analysis by TAI, U.S. Air Force (USACE), FAA, and U.S. Navy. Limited to flexible pavement systems, it is an in-situ destructive test that can be used on all unbound pavement layers. The details of in-situ and undisturbed sample CBR testing may be found in Military Standard MIL-STD-621A (Test Method 101).

Table 1. Summary of overlay categories by agency.

Overlay	Existing Pavement	System Behavior	AGENCY (USACE) USAF	CDOT	USN	FAA	TAI	PCA
Non-Rigid	Flexible		X	X	X	X	X	
Non-Rigid	Composite						X	
Non-Rigid	Rigid	Rigid	X	X	X	X	X	
Non-Rigid	Rigid	Flexible		X	X		X	
Rigid	Flexible		X		X	X		X
Rigid	Rigid	Rigid (Bond)			X			X
Rigid	Rigid	Rigid (Partial Bond)	X	X	X	X		X
Rigid	Rigid	Rigid (Unbonded)	X	X	X	X		X
Rigid	Composite	Flexible	X	X				X

Figure 4. Methods of overlay design approach.

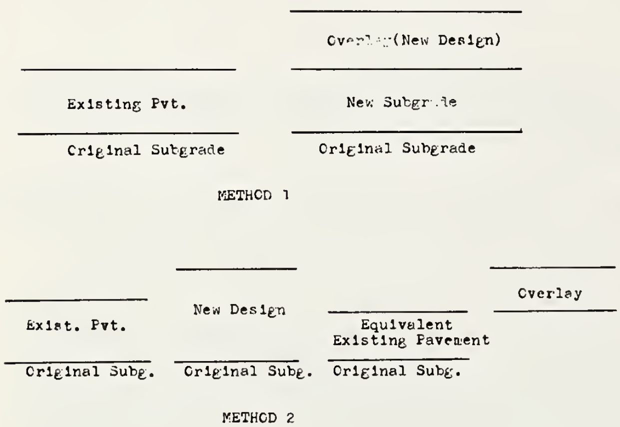


Table 2. Design approaches by overlay category.

Overlay	Existing Pavement	System Behavior	Overlay Design Method (1)	Overlay Equation
Non-Rigid	Flexible		2	$t_o = t_n - t_e$
Non-Rigid	Composite		2	$t_o = t_n - t_e$
Non-Rigid	Rigid	Rigid	2	$t_o = 2.5\,(Fh_n - h_e)$ (2)
Non-Rigid	Rigid	Flexible	2	$t_o = t_n - t_e$
Rigid	Flexible		1	$h_o = h_n$
Rigid	Rigid	Rigid (Bond)	2	$h_o = h_n - h_e$
Rigid	Rigid	Rigid (Partial Bond)	2	$h_o = \sqrt[1.4]{h_n^{1.4} - Ch_e^{1.4}}$ (3)
Rigid	Rigid	Rigid (Unbonded)	2	$h_o = \sqrt[2]{h_n^{2} - Ch_e^{2}}$
Rigid	Composite	Flexible	1	$h_o = h_n$

Notes:

(1) Method 1 - Design subgrade surface of existing pavement.
Method 2 - Use existing subgrade to design new-slab.

(2) Except USN, TAI.

(3) CDOT: $h_o = \sqrt{h_n^{1.87} - Ch_e^{2}}$

(4) General Nomenclature

h refers to rigid pavement thickness

t refers to flexible pavement thickness

o refers to overlay pavement thickness

n refers to new pavement

e refers to existing pavement

Resilient Modulus Test - As defined by TAI, the resilient modulus, E_S or M_R, is the elastic (recoverable) modular response obtained from repeated load triaxial testing. The M_R value is

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

where σ_d = applied repeated deviator stress and
e_r = elastic axial strain. The M_R value is used as the primary test procedure for existing subgrades by TAI for all non-rigid overlays. Because the procedure uses triaxial apparatus, the test cannot be conducted in-situ. For overlay analysis, undisturbed samples are required of the existing subgrade.

The specific test procedure (127) is only applicable for untreated fine-grained subgrade soils. Although procedures for M_R testing of unbound granular materials are available in the literature (92), the test procedure recommended by TAI is not intended to account for the stress dependency on M_R results for coarse-grained subgrades or unbound subbase-base course materials.

For fine-grained soils the stress conditions selected are intended to approximate in-place stress states for moving wheel loads. The design condition for the repeated load test is σ_c = 2 psi and σ_d = 6 psi. Sample conditioning is accomplished after a σ_c = 2 psi value is initially applied to the specimen. Deviator stress conditioning levels of 3, 6 and 9 psi are applied for approximately 200 load cycles each. Upon completion of conditioning, the σ_d is reduced from 9 to 6 psi, and the e_r after 200 repetitions is used to compute the M_R value. Approximations of the M_R value may be made, in TAI manual, from (a) CBR correlation established by Shell investigators (M_R=1500 CBR) (83), (b) correlation to the FAA -F soil subgrade category ratings, and (c) correlation to plate load subgrade support values "S" (30 inch diameter plate, Δ = 0.5 inch and n = 10 repetitions).

Subgrade Classification Ratings - The method of rating subgrade soils for structural design requirements for both rigid and flexible pavement systems is used exclusively by the FAA procedure. The procedure is based on 11 subgrade classes for flexible pavement evaluation (Fa to F10) and 5 categories of rigid pavement design. The subgrade class is selected by FAA soil group (E rating) as well as drainage and frost conditions. Although this system is still present in change 4 of AC150/5320-6A, new design curves present for wide-body heavy aircraft - introduced in change 4 - are exclusively based on measured modulus of subgrade reaction values (k) for rigid pavement designs and not the R subgrade classification. Flexible pavement design curves for these new aircraft are based on the F ratings. In this regard, FAA allows designs to be based on CBR values by correlation of the CBR to F soil rating (39).

Plate Load Tests

Three types of plate load tests are used by various agencies.

Soil Support Value - The use of the soil support value, S, from plate load tests was introduced by McLeod and the Canadian Department of Transportation more than 25 years ago (86). The S value itself is used for flexible pavement analysis and represents the subgrade support, in pounds, that will be supported by a subgrade under a 30-inch diameter plate, specified deflection level, usually Δ = 0.5 inch and n = 10 repetitions of load. The test procedure followed by the CDOT is ASTM D-1195, a repetitive plate load test procedure. Three load levels are used to apply deflections of 0.05, 0.20, and 0.40 inch with each load being applied and released 6 times before going to the next load increment (68).

One important feature of the plate load test used for flexible pavement analysis is that through the use of the basic CDOT design equation,

$$t = K \log \frac{P}{S}$$

the S value may be determined either directly at the subgrade of an existing pavement or from plate load tests conducted at the pavement surface. In the above equation, t is the equivalent thickness of unbound granular material, k is the base course constant, which is an inverse measure of the supporting value of the base course per unit of thickness, P is the gross single wheel load to be carried on the runway, and S is the subgrade support at the same Δ, n and A_C (or plate diameter), that apply to P.

By conducting plate load tests on the surface of an existing pavement, P is directly found from the test. If the thickness and composition of the existing pavement layers are known, the equivalent t may be calculated by CDOT conversion factors (noted in next section) and the S value may be computed from:

$$S = \frac{P}{\text{antilog } \frac{t}{k}}$$

In the CDOT overlay analysis, options are also available to use load data from other sites having similar soil, climatic, and drainage conditions. If this cannot be determined, S values may be estimated from soil classifications that have been developed. If the modulus of subgrade reaction, k, is known for a particular site, the subgrade support value may be determined from

$$S = \frac{k}{0.00777}$$

Pavement Support Value - Previous flexible pavement designs by the U.S. Navy have used plate load results with Burmister's two-layered theory. However, the most recent change in DM-21 (36) has eliminated this test as a method of flexible pavement design and relies exclusively on the CBR design procedure. The use of the plate load test for evaluation and overlay design of non-rigid over flexible pavements is still used.

The plate load test procedure for the U.S. Navy is ASTM D-1196. When used on non-rigid overlay analysis for existing flexible pavements, 8 and 30-inch diameter plates are used at 3 test locations (surface tests). Each test is conducted until a deflection, $\Delta = 0.2$ inch, is attained or the capacity of the reaction load is reached. At one of the three locations, a 4 x 4 foot pit is excavated to a depth of approximately 10 inches (This depth should be conducted at the bottom of the high quality base course.) Tests using a 30-inch diameter plate are conducted at this location and then at the top of the subgrade layer.

The results are subsequently plotted on a P (Δ=0.15") versus plate diameter scale for the overlay analysis. Such a diagram is shown in Figure 6. For the design or critical aircraft, the design wheel coordinates (P,p) are located, and a line is drawn parallel through this point to the line connecting the surface support values, P, determined from the 8 and 30-inch diameter plate. The required overlay thickness may be read directly from this figure.

Modulus of Subgrade Reaction - The classical modulus of subgrade reaction, k, is used by all agencies for overlay analysis except TAI. The k value is a measure of the deflection response of a pavement layer to applied pressure. As such it can be computed by finding the deflection at a given pressure (e.g., USACE: for p - 10 psi) or the pressure necessary to cause a certain deflection (e.g., PCA: p for $\Delta = 0.05$ inch).

The test procedure specified by the FAA and USAF (USACE) is MIL-STD-621 (Method 104), whereas ASTMD-1196 is followed by PCA and the USN. The CDOT uses the results of the procedure used to evaluate S and the correlation equation:

$$k = 0.00777S$$

One important difference between k measurements and the procedures used to establish S and P is the fact that k values are normally based on non-repetitive load tests, while the latter are based on repetitive tests. Because almost all of the design methods discussed are empirical in nature (methods have been related to field performance), it is of the utmost importance to use the specific test procedure specified for each design case.

For any type of plate load testing (S, P, or k), comparison of values is meaningless without noting the plate diameter, number of load repetitions, and the deflection at which the value is evaluated. In this regard, it is interesting to note that, when Westergaard developed his dense liquid interior load theory for rigid pavements in 1926, he never defined how this parameter could be measured but knew it was a function of the loaded area. He indicated that, as long as the analysis was limited to a particular type of load (wheel load on pavement), empirically developed k values would be sufficient for the analysis.

In 1941, the USACE conducted an analysis at Wright Field to check the validity of Westergaard's interior load case (115). Of importance was the fact that plate sizes from 12 to 72 inch diameter were used to test predictability of stresses to k values determined by volumetric slab displacements caused by the wheel loads. It was found, for the wheel load conditions of the test, that the 30-inch diameter plate gave the best comparative results.

Several significant changes in airfield pavement conditions have prompted some thoughts regarding the adequacy of this test condition. The use of stabilized subbases has increased, and the

Figure 5. U.S. Navy flexible overlay on rigid pavement.

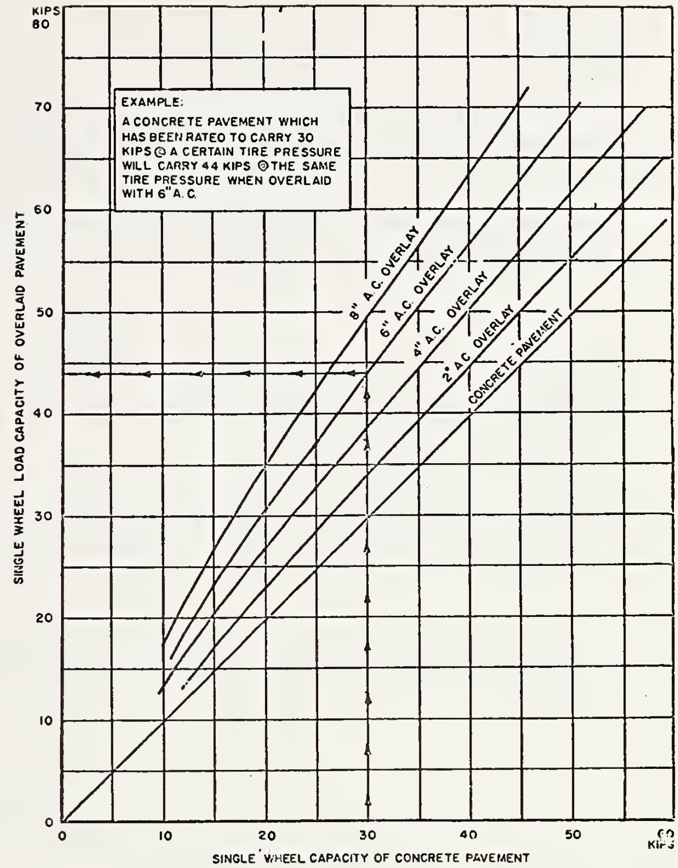
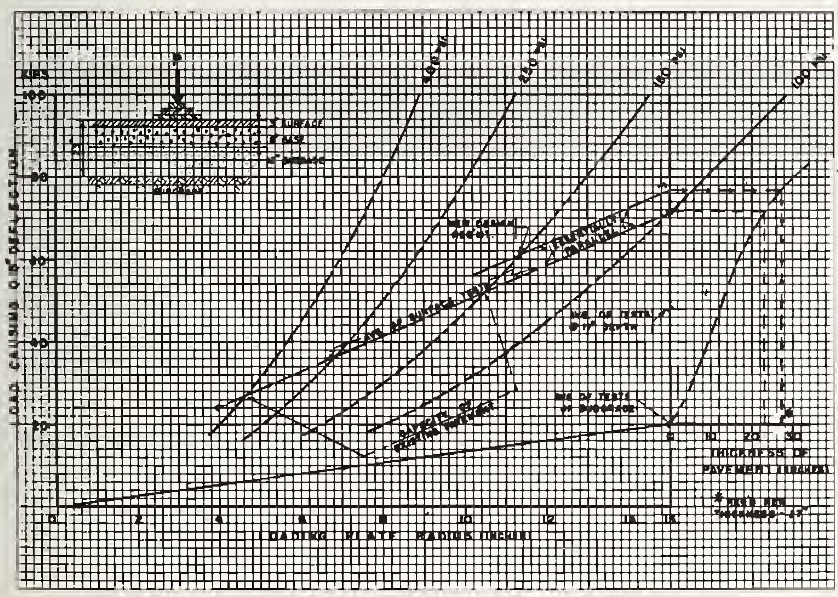


Figure 6. U.S. Navy load test method of pavement design.



size of the loaded area for wide-bodied, multiple wheel aircraft of today frequently occupies an entire slab.

Though the primary reason for subbases in rigid pavements is not due to increasing the effective k value of the layered system, the use of stabilized subbase obviously contributes to a higher k value. This concept is not currently recognized in design (new or overlay analysis) because most agencies specify a maximum k value of 500 to 1,000 pci.

The adjustment due to multiple wheeled aircraft rigid pavement design has been recognized only by the PCA. The procedure used by the PCA is to adjust the radius of relative stiffness value, l,

$$l = \sqrt[4]{\frac{E_1 h^3}{12k(1-\mu^2)}}$$

of the pavement system. It is of interest to note that this adjustment has been based on the use of multi-layered elastic theory.

Modulus of Rupture - The flexural strength of PCC pavements is found by modulus of rupture MR, tests, generally in accordance with ASTM C-78 or C.E. Standard CRD-C-16. All agencies except TAI require this test for both new (rigid overlay) and existing rigid pavements to be overlaid. The MR value used in analysis is the 28 day strength by the USN and CDOT methods, while the 90 day strength (normally 100-114 percent of 28 day strength) is specified for FAA, PCA, and USAF.

A special problem encountered in rigid over rigid overlays (unbonded and partially bonded) concerns the frequent occurrence where the new overlay has an MR value different from the existing rigid pavement. In this situation, an adjusted or modified h_e value is used in the overlay design equation.

$$he = \left(\frac{h}{h_{dc}} \times h_c\right)$$

where h is based on the 90 day flexural strength of the new pavement and h_{db} is based on the flexural strength of the existing pavement.

Whenever possible, all agencies generally recommend the testing of sawed beam samples for MR evaluation. However, several correlations to other tests, such as compressive strength ($MR = k\sqrt{f'_c}$ where k is a constant between 8 and 10) and splitting tensile test ($MR = T + 200$ where T is the split tensile strength from ASTM C-498). For the procedure used by the CDOT, minimum strength values of 550 psi and a working stress of 460 psi are required. No decrease in PCC thickness is taken into account for higher MR values.

Condition and Equivalencies of Existing Layers

A need exists to determine the thickness of the existing pavement (t_e or h_e), in terms of equivalent material, which is the same as that found for the new pavement thickness (t_n or h_n). The type of equivalent material specified depends on whether a flexible or rigid overlay is being used and the agency selected. For example, the approach used by TAI is based on an equivalent thickness of high quality asphalt concrete mix for all non-rigid overlay analysis. The equivalent material type used by USACE, FAA, and CDOT in non-rigid overlays is an unbound non-stabilized granular material. For any rigid over rigid overlays the equivalent material is PCC.

To determine the equivalent thickness of the existing pavement, one must account for differences in material types by layer equivalencies and the condition of each layer by condition factors which result in an effective thickness of the equivalent material type. This concept is used in both rigid and non-rigid overlay analysis by a series of conversion factors. As used in this report, the conversion factor thus accounts for differences in both layer material and the condition of the layer.

A summary of all the conversion factors that are used by each agency in overlay analysis are shown in Tables 3 to 8.

Table 3. U.S. Air Force conversion factors.

MATERIAL TYPE	EQUIVALENCY
(1) 1" A. C.	1" G. B.
(2) 1" P. C. C.	1" G. B. (CBR ≥ 80)
(3) 1" P. C. C. (good condition)	1.0" P. C. C.
(3) 1" P. C. C. (initial corner cracking, no progressive cracking)	0.75" P. C. C.
(3) 1" P. C. C. (badly cracked or crushed)	0.35" P. C. C.

Notes: (1) Used for all Non-Rigid Overlays

(2) Used for Rigid Over Composite (Flexible System)

(3) Used for Rigid Over Rigid Overlays

Table 4. Canadian conversion factors.

MATERIAL TYPE	EQUIVALENCY
1" P. C. C. (Good Condition)	3.0" Gran. Base (G. B.)
1" P. C. C. (Fair Condition)	2.5" G. B.
1" P. C. C. (Poor Condition)	2.0" G. B.
1" A. C. (High Quality)	2.0" G. B.
1" Asphalt Pavement	1.5" G. B.
1" Water Bound Macadam Base Course	1.5" G. B.
1" Crushed Gravel	1.0" G. B.
1" Crushed Stone Base	1.0" G. B.
1" Granular Base	1.0" G. B.
*1" P. C. C. (Good Condition)	1.0" P. C. C.
*1" P. C. C. (Initial corner cracks, no progressive cracking)	0.75" P. C. C.
*1" P. C. C. (Badly cracked or crushed)	0.35" P. C. C.

* Used for Rigid Over Rigid Overlays

Table 5. U.S. Navy conversion factors.

MATERIAL TYPE	EQUIVALENCY
(1) 1" A. C.	1" G. B.
(2) 1" P. C. C.	2" G. B.
(3) 1" P. C. C. (Good Condition)	1.0" P. C. C.
(3) 1" P. C. C. (Initial corner cracks, no progressive cracking)	0.75" P. C. C.
(3) 1" P. C. C. (Badly cracked or crushed)	0.35" P. C. C.

Notes: (1) Used for all Non-Rigid Overlays

(2) Used for Non-Rigid Over Rigid (Flexible System)

(3) Used for Rigid Over Rigid Overlays

Table 6. Federal Aviation Administration conversion factors.

MATERIAL TYPE	EQUIVALENCY
(1) 1" AC. C. Surface (P-401, good condition, bit. overlay)	1.5" G. B.
(1) 1" A. C. Surface (P-401, poor condition)	1.0" G. B.
(1) 1" A. C. Base (P-201, good condition, bit. overlay)	1.5" G. B.
1" C. T. B. (P-304, good condition)	1.5" G. B.
(2) 1" Granular Base	1.5" G. B.
(3) 1" P. C. C. (good condition)	1.0" P. C. C.
(3) 1" P. C. C. (initial corner cracking, no progressive cracking)	0.75" P. C. C.
(3) 1" P. C. C. (badly cracked or crushed)	0.35' P. C. C.
(4) 1" A. C. (P-401 or P-201, good condition)	1.5" S. B. (Rigid Pvt.)
(4) 1" Granular Base	1.0" S. B. (Rigid Pvt.)

Notes: (1) Used for Non-Rigid Overlays.

(2) Only substition made if flexible pavement subbase less than 3".

(3) Used for Rigid Over Rigid Overlays.

(4) Only additions or changes from Change 4, FAA Manual. Applicable only to Rigid Over Flexible Overlays and considers equivalency of existing flexible pavement to minimum rigid pavement subbase requirements.

Method 1 Designs

The overlay design philosophy characterized by method 1 approaches involves the concept that the existing pavement is treated as a new subgrade (base) for the new overlay. As noted in Table 2, this method is used for rigid over flexible overlays and rigid over composite overlays acting as a flexible system.

Rigid Over Flexible - As shown in Table 1, this overlay category is used by USACE, USN, PCA, and FAA. The methods used by the USAF, USN, and PCA require that field tests be made to determine the k of the existing pavement. The procedure of the present FAA system relies on classifying the existing subgrade as to its R rating (Ra to Re) and viewing the flexible pavement as being a rigid pavement subbase. In the proposed change 4, the same procedure is used plus an alternative of direct k measurement on the surface of the existing pavement. In the FAA procedure, particular importance is placed on evaluating the existing flexible pavement relative to the minimum subbase requirement. If these requirements are not met, reductions in k or increases in h_n must be made for the overlay analysis.

Therefore, with the exception of the FAA procedure, all other agencies specify direct tests to evaluate k. All agencies, however, specify that the k_{max} = 500 pci. (Although this value is not specifically noted in the PCA method, it does note this limiting k_{max} value by other agencies and points out that this limitation is somewhat arbitrary. PCA cites the need for more developmental work to realize fully the advantages of composite designs and when stabilized subbases may be used.)

In addition to the k_{max} = 500 pci restriction, USACE procedure also notes that

$$k_n = k_s + (k_s - 25) N$$

where k_n = subgrade reaction value of the existing structure, k_s = existing subgrade k value, and N = number of 6-inch layers of high quality base present in the existing flexible pavement structure. Using this concept, an additional restriction that $k_n - k_s \leq 125$ pci is placed on the analysis.

Rigid Over Composite (Flexible) - The agencies specifying this type of overlay are USACE, CDOT, and PCA. However, there is no agreement between agencies on what differentiates a rigid over composite flexible system from a rigid system. USACE recommends that thickness of the separating course greater than 4 inches constitutes a flexible system; the CDOT recommends greater than 6 inches, while no specific thickness recommendations are made by the PCA other than that if the separating course thickness is large enough, this condition will occur.

The restrictions and procedures recommended by the USAF are identical to those noted for the rigid over flexible condition. In addition, the presence of the existing PCC layer is treated as a high quality base in determining the $k_n = k_s + (k_s - 25)N$ equation. The k_{max} = 500 pci specified by the USAF conflicts with the k_{max} = 1,000 pci recommended by the CDOT for this type of pavement construction. In the CDOT method, both direct and indirect procedures may be used to determine the k of the existing pavement. The direct approach uses the measurement of P at the surface with the correlation equation

$$k = 0.00777P$$

The indirect approach uses knowledge of the existing S of the subgrade along with the design equation

$$P = S \text{ antilog } \left(\frac{t}{k}\right)$$

Knowing S, P can be computed from the equation and then k can be calculated from the previous equation.

The procedures used by the PCA is to directly determine the k value from field tests on the existing pavement structure.

Use of Conversion Factors - For the two overlay categories discussed, it can be seen that the majority of agencies specify that k values be determined on top of the existing surface and place a maximum limit on the k to be used for the overlay analysis. For these procedures, involving direct k tests, no condition or layer equivalency factors (i.e., conversion factors) are required because the features of condition and material type are directly incorporated into the measured k response. When procedures are specified that allow other than direct testing of the pavement k, conversion factors must be used. For example, if the CDOT method is used involving knowledge of the S value, the thickness t used in

$$P = S \text{ antilog } \left(\frac{t}{K}\right)$$

must be expressed in terms of the equivalent thickness of granular material. Referring to Table 4 shows conversion factors that account for layer equivalencies and condition of the existing PCC pavement relative to a granular base.

The same concept is employed by FAA for rigid over flexible analysis as shown in Table 6 (note 4). Conversion factors are used to convert the existing flexible pavement into equivalent rigid pavement subbase requirements.

Method 2 Designs

In contrast to the prevalent concept of method 1 designs that do not rely on conversion factors, the approach philosophy of method 2 designs takes into account both the material type effect and layer condition to arrive at the existing pavement thickness (either t_e or h_e). In this design method approach, the use of conversion factors is the rule rather than the exception. The discussion of the various procedures used by each agency in method 2 designs is best explained by subdivision on the basis of overlay type categories.

Rigid Over Rigid Overlays - Because the overlay category implies that the new overlay and existing pavement are of the same material, there is no need to use layer equivalencies between materials. From Table 2, the design equations for the various conditions of bond between layers are

$$h_o = h_n - h_e \text{ (bond)}$$

$$h_o = \sqrt[1.4]{h_m^{1.4} - Ch_e^{1.4}} \qquad \text{(partial bond)}$$

$$h_o = \sqrt[2]{h_m^{2} - Ch_e^{2}} \qquad \text{(unbonded)}$$

Because layer equivalencies are unnecessary, the conversion factors for these design overlay situations only involve the use of pavement condition factors. This is accomplished by C values of the overlay equations noted. All agencies specifying these overlay categories use the same C values for various slab conditions.

Each category of bond available controls the applicability of the equation to the initial C value of the existing slab. For example, the use of a bonded overlay is only recommended for existing slabs that possess a C factor generally greater than 0.90. Because the C value must approach unity for these conditions, no C factor is necessary in the bond equation.

The partial bond overlay is generally recommended for structural slab conditions of the existing pavement that are generally significantly better than a C = 0.35 condition. If a slab is badly shattered, a bonded or partially bonded overlay may not be used. The only type of rigid over rigid overlay that is suggested for this type of slab is the unbonded case.

Finally, because of the above restrictions of overlay bond conditions relative to the existing structural slab condition, the use of bonded overlays is strictly limited to overlays for strengthening rather than complete rehabilitation purposes. On the other hand, unbonded overlays may be applicable to either case. Thus one may view the selection of overlay type in terms of the remaining damage, D_R, of the existing slab by the selection of the conversion factor.

Non Rigid Overlays - In non-rigid overlay analysis, conversion factors are necessary for layer material and condition factors of the existing pavement and may be used as layer equivalencies to convert the calculated overlay thickness of an equivalent material into various combined layer materials.

In the procedure recommended by the USAF for non-rigid over flexible overlays, the equivalent material is unbounded (non-stabilized) granular material. No differences between unbound granular material and stabilized layers are recognized. This implies that the conversion factors for these situations are unity between material types (Table 3, note 1). Because of this feature, no pavement condition factors are used in obtaining equivalent thicknesses of existing flexible pavement or reduction in overlay thickness to account for variations in material quality.

For CDOT, the equivalent material is unbound granular material. Employing either approach (measure P at surface and calculate S or measure S directly), conversion factors are used for both layer equivalencies and condition (Table 4). Based on the original work of McLeod (86), no difference was found between crushed stone base and granular subbase from in-place measurements. Further, 1 inch of high quality asphalt concrete mix was equivalent in load support capacity to 2 inches of unbound granular material. The conversion factors used are applicable for both evaluation of the existing flexible pavement and converting the overlay thickness into various layers.

The equivalent material used by the USN procedure for non-rigid over flexible overlays is high quality asphalt concrete (AC). However, no allowance or difference is made between stabilized and unstabilized materials. Hence, the conversion factors shown in Table 5 reflect a 1 to 1 equivalency between AC and GB similar to the USAF (USACE). Because plate load tests are used directly on the surface, base, and subgrade of the existing flexible pavement, no condition factors are shown nor required by the USN procedure for this overlay type because the condition is directly accounted for in the pavement response to the plate load tests. The procedure recommended by the FAA is similar to that of the CDOT in that conversion factors reflect differences in both material type and layer condition. The conversion factors are shown in Table 6. In contrast to the USAF and USN procedures stabilized materials (both AC and CTB) are assigned equivalencies that reflect superior load capacities relative to unbound materials.

In the procedure recommended by The Asphalt Institute, the equivalent material is AC. The conversion factors recommended reflect a wide range of material equivalencies and condition factors (Table 7). Because of the range in material-condition combinations shown, the overlay procedure is applicable for almost any existing pavement structure (flexible, composite, rigid-rigid systems, and rigid-flexible systems).

Non-Rigid over Composite - TAI is the only agency specially noting this type of overlay design. The procedure and comments are identical to the non-rigid over flexible design.

Non-Rigid Over Rigid (Rigid System) - The common overlay design approach for this overlay category is based on field tests and performance studies conducted by the USACE. The overlay design equation

$$t_0 = 2.5\ (Fh_n - h_e)$$

is used by USACE, CDOT, and FAA. The existing rigid slab is presumed to be in good condition. Thus the C factor of the existing slab approaches unity and, like the case for bonded rigid over rigid overlays, is generally neglected.

It should be noted that the equivalent material type used for this equation solution is based on PCC though the overlay is non-rigid. However a factor of 2.5 represents a layer equivalency of 1 inch PCC equivalent to 2.5 inches of non-rigid overlay. Figure 7 represents the original data plot used by the USACE in developing this overlay design equation. One of the major conclusions drawn from the USACE study indicates:

"There is no great difference between overlays incorporating high quality base course materials and those constructed of asphaltic concrete for their full depth" (90).

The F Factor shown in the equation is also an extremely important observed feature from the field test program. One of the major conclusions from the study showed that, though initial cracking of the existing PCC slab occurred in relative agreement with new design theory, the subsequent rate of crack propagation of the slab was a distinct function of the property of the subgrade soil, with weak subgrades showing a higher rate of crack growth than stronger granular subgrades. Figure 8 shows this relationship from the CE performance studies. It can be seen that, for any selected value of design coverages (repetitions) to failure, a plot of the percent thickness design versus subgrade reaction k results. This relationship is exactly what is used by USAF, CDOT, and FAA. Therefore the F factor is really a thickness design percentage.

Figure 9 illustrates the F relationship used by the USAF. As can be seen, several curves are present for various design coverage levels. Figure 10 represents the relationship used by the CDOT and is typified by the capacity traffic design case previously discussed. Table 9 is the F values used by the FAA. Because the rigid pavement design analysis involves use of R ratings for subgrade categories, this factor is presented in tabular form.

The procedures recommended by TAI have been previously discussed while the procedure for this overlay type used by the USN has been shown in Figure 5. In the latter method it is to be noted that there is no condition factor present for the existing PCC slab in DM-21 and it is therefore

Table 7. Asphalt Institute conversion factors.

MATERIAL TYPE	EQUIVALENCY
Asphalt Materials	
1" A.C. (uncracked, little deformation)	0.9" - 1.0" A.C.
1" Asphalt Treated Base (other than A.C.)	0.7" - 0.9" A.C.
1" Liquid Asphalt Mix (stable, uncracked, [illegible] ef.)	0.7" - 0.9" A.C.
1" A.C. (fine cracking, slight deformation, stable)	0.7" - 0.9" A.C.
1" A.C. (appreciable cracking, little or no spalling, some deformation, but essentially stable)	0.5" - 0.7" A.C.
1" A.C. (large cracks, spalls, appreciable deformation)	0.3" - 0.5" A.C.
Portland Cement Concrete	
1" P.C.C. (stable, uncracked)	0.9" - 1.0" A.C.
1" P.C.C. Base under A.C. (stable, little cracking)	0.9" - 1.0" A.C.
1" P.C.C. (stable, slight cracking)	0.7" - 0.9" A.C.
1" P.C.C. (appreciably cracked, faulted, fragments 1-4 yd.^{2})	0.5" - 0.7" A.C.
1" P.C.C. (Broken, pieces 2' or less in max. dimensions - with subbase)	0.3" - 0.5" A.C.
1" P.C.C. (Broken, pieces 2' or less in max. dimensions - on subgrade)	0.3" - 0.5" A.C.
Cement Stabilized	
1" Soil Cement Base (little cracking, stable surface)	0.5" - 0.7" A.C.
1" Soil Cement Base (extensive pattern cracking, unstable)	0.3" - 0.5" A.C.
1" Cement Modified Base/Subbase (Soil PI ≤10)	0.2" - 0.3" A.C.
Unbound Granular	
1" Granular Base (Nonplastic, high quality)	0.3" - 0.5" A.C.
1" Granular Base or Subbase (CBR >20, PI ≤ 6)	0.2" - 0.3" A.C.
1" Granular Base or Subbase (CBR >20, PI > 6)	0.2" - 0.3" A.C.

* Conversion Factors applicable only to pavement evaluation for overlay design.

Table 8. PCA conversion factors.

MATERIAL TYPE	EQUIVALENCY
*1" P.C.C. (good condition)	1.0" P.C.C.
*1" P.C.C. (initial corner cracks no progressive cracking)	0.75" P.C.C.
*1" P.C.C. (badly cracked or crushed)	0.35" P.C.C.

*Used for Rigid Over Rigid Overlays

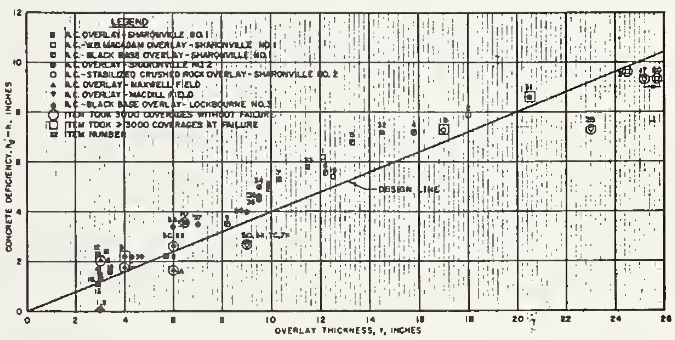


Figure 7. Concrete deficiency versus nonrigid overlay thickness.

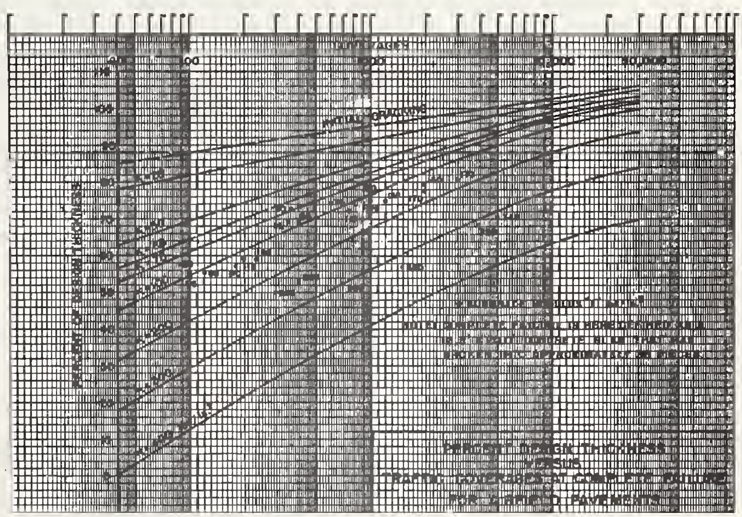


Figure 8. Percentage of thickness versus coverages to complete failure.

presumed to be valid only for slabs in good condition.

Several alternative overlay approaches are suggested by FAA and USACE for this design situation. The proposed change 4 of the FAA manual recommends an alternate design equation:

$$t_0 = 2.5\ (0.8h_n - h_e)$$

where h_e is found strictly by evaluation methods outlined in Chapter 6 of the manual. In this situation the word evaluation is meant to imply the load capacity evaluation of the existing pavement slab. Because of certain reductions in the load carrying evaluation that may be necessary because of subbase requirements, the h_e determined by this procedure does not have to be equal to the existing pavement thickness.

The USAF (USACE) also recommends that a flexible over flexible overlay design check be made of this type of overlay category. In this procedure the existing PCC slab is assumed to be equivalent on an inch for inch basis with high quality base course material. Because of the system of conversion factors used by TAI, it is not necessary to determine under what conditions a non-rigid over rigid overlay behaves as a flexible system or a rigid system. The CDOT however states that, if the t_o value is less than 10 inches on a slab that is not shattered or is less than the existing slab thickness, a rigid system occurs. The USN specifies the t_o value must be less than 8 inches. No such information is noted for either FAA or USAF approaches.

Non-Rigid Over Rigid (Flexible System) - The agencies noting this overlay category are the USN, CDOT, and TAI. The CDOT analysis for this overlay is the same as that for the non-rigid over flexible overlay and the comments made are applicable to this system. TAI analysis has been previously noted as identical for all non-rigid overlays. The only different approach for this overlay is recommended by the USN. For this case the CBR of the existing subgrade is determined for use in finding the t_n. The existing rigid pavement slab is converted to equivalent granular material by the layer equivalency shown in Table 5 (note 2). No facotr is suggested in DM-21 to account for the condition of the existing rigid pavement.

Agency Differences in Non-Rigid Overlays - Table 10 is a summary showing agency differences in the types of non-rigid overlays recommended, layer equivalencies recommended, and whether unbound subbases may be used in the overlay.

Both types of non-rigid overlays (bituminous or flexible) are recommended by the USAF, CDOT, and the present FAA procedure, while only bituminous overlays are recommended by TAI, USN, and change 4 of the FAA procedure. Of the agencies allowing flexible or sandwich overlays, the USAF allows subbase material to be used only for the non-rigid over flexible overlay, whereas the CDOT allows subbase for any type of non-rigid condition. The differences in layer equivalence between AC and unbound granular material are of course self evident and vary from the one to one equivalence for the USAF to the two to one equivalence of the CDOT method.

MAJOR RESEARCH IN PROGRESS

In any fundamental approach to new pavement or overlay analysis one must consider (a) the theory used to define critical parameters, (b) defining the failure criteria in terms of the critical parameters, and (c) measuring the required material properties that are necessary input for the theory. In general, recent pavement research in airfield overlay analysis has attacked each factor with equal vigor.

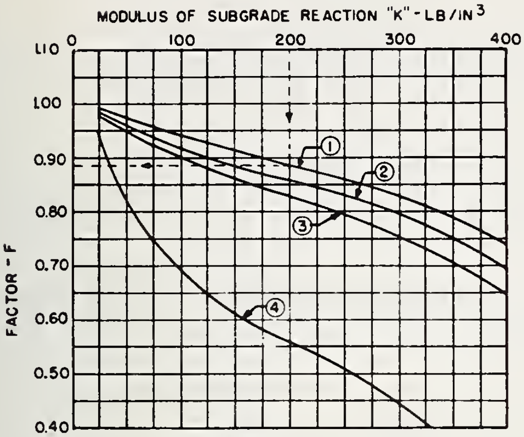
As can be seen from the review of existing overlay theories, design concepts have been based on empirical methods for flexible pavements and more theoretical approaches for rigid pavement designs. However, the lack of a proven theoretical treatise to incorporate stabilized subbase, complex composite pavements, and new overlay materials, such as fibrous concrete (50), is causing airfield engineers to reevaluate concepts that have been proved in the past for rigid pavements and overlays.

Some of the major concepts generated by research efforts over the last decade deserve further review.

Energy-Transfer Function Approach

One of the most recent and innovative research concepts developed for overlay analysis is based on the work of Harr and associates (8, 10, 54, 55). The hypothesis of this effort is that:

Figure 9. U.S. Air Force overlay design procedure for nonrigid overlays.



LEGEND

CURVE 1 - USE FOR TYPE A TRAFFIC AREA FOR ALL MULTIPLE WHEEL GEAR DESIGNS EXCEPT TWIN-TWIN WHEEL GEAR.

CURVE 2 - USE FOR TYPE A TRAFFIC AREA FOR TWIN-TWIN WHEEL GEAR DESIGN.

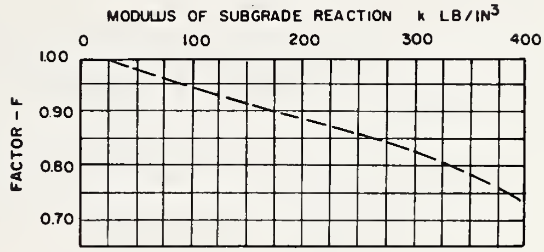
CURVE 3 - USE FOR TYPE B AND C TRAFFIC AREAS FOR ALL GEAR DESIGNS.

CURVE 4 - USE FOR TYPE D TRAFFIC AREA FOR TWIN-TWIN WHEEL GEAR DESIGN ONLY.

NOTES:

(1) DETERMINE FACTOR, F, FROM APPLICABLE CURVE ABOVE.
(2) DETERMINE PLAIN RIGID PAVEMENT THICKNESS, h_d, FROM APPLICABLE DESIGN CURVE, FIGURE 1 THRU 13 USING FLEXURAL STRENGTH OF EXISTING PAVEMENT AND MEASURED SUBGRADE MODULUS, "K".
(3) DETERMINE NON-RIGID OVERLAY THICKNESS AS FOLLOWS:
$t = 2.5\,(Fh_d - h)$
WHERE: t = REQUIRED NON-RIGID OVERLAY, INCHES
h = EXISTING RIGID PAVEMENT THICKNESS, INCHES
(4) MINIMUM THICKNESS OF NON-RIGID OVERLAY = 4 INCHES.
(5) MINIMUM VALUE OF F = 0.40. FOR K > 400 USE F FOR K = 400.

Figure 10. CDOT factor for nonrigid overlays.



NOTES:

(1) DETERMINE FACTOR, F, FROM CURVE ABOVE.
(2) DETERMINE PLAIN RIGID PAVEMENT THICKNESS, h_d, FROM APPLICABLE DESIGN CURVE AND MEASURED SUBGRADE MODULUS "k" (Top of Base).
(3) DETERMINE NON-RIGID OVERLAY THICKNESS AS FOLLOWS:

$$t = 2.5\,(Fh_d - h)$$

WHERE: t = REQUIRED NON-RIGID OVERLAY, INCHES
h = EXISTING RIGID PAVEMENT THICKNESS, INCHES
(4) 1 INCH ASPHALTIC CONCRETE EQUALS 1½ IN. OF NON-RIGID OVERLAY.
(5) k > 400 USE F FOR k = 400.

Table 9. FAA F-value for nonrigid over rigid overlays.

EXISTING SUBGRADE CLASS	Ra[1]	Rb	Rc	Rd	Re
	Values of F when subbase under existing pavement conforms to requirements for class of subgrade below.				
Ra	0.80				
Rb	0.90	0.80			
Rc	0.94	0.90	0.80		
Rd	0.98	0.94	0.90	0.80	
Re	1.00	0.98	0.94	0.90	0.80

(1) Figures in this column apply when no subbase has been provided.

Table 10. Nonrigid overlay differences by agency.

FACTOR	AGENCY USAF	CDOT	TAI	(Present) FAA	(Change 4) FAA	USN
1. Types of Non-Rigid Overlay Allowed	Bit., Flex.	Bit., Flex.	Bit.	Bit., Flex.	Bit.	Bit.
2. Unbound Subbase Material Allowed	Yes (1) No (2)	Yes	No	No	No	No
3. Layer Equivalencies	1"AC=1"GB	1"AC=2"GB	NA	1"AC=1.5"GB	1"AC=1.5"GB	NA

Notes: (1) Non-Rigid Over Flexible

(2) Non-Rigid Over Rigid

"A functional relationship exists between the cumulative energy, as measured by cumulative peak deflections imparted to a given pavement system, and the condition of that system" (54).

Analysis of AASHO Road Test findings was conducted and a typical result for several AASHO structures is shown in Figure 11. The asymptotic relations of these curves as failure is reached can be seen. Using regression analysis of load test findings, an equation was developed relating present serviceability index PSI to thicknesses of each component layer and the cumulative total peak deflection

$$PSI = 0.031 + 0.383(S) + 0.077(B) + 0.071(SB) - 0.0022(D) + 5.56\text{x}10^{-7}(D^2)$$

where S. B. SB, represent the layer thicknesses in inches and D is the cumulative total pavement deflection in feet.

The concept is based on the philosophy that the induced energy of the load (wheel of prime mover) is available to do the work of deflecting the pavement system. The total strain energy of the system can be shown as

$$U_t = \frac{EI}{2}\int_{-\infty}^{+\infty}\frac{d^2D}{dx^2}\ DX + K\int_{-\infty}^{+\infty}(W\quad x\quad)^2 dx$$

where

D = surface deflection,
W = subgrade deflection,
x = horizontal distance from the load centerline, and
k = stiffness of a Winkler foundation.

The first part is representative of the strain energy due to the pavement bending and the second is the strain energy associated with deformation of the foundation. The assumption is made that, because both terms are related to deflection, this parameter is critical to performance.

Using the concepts noted, two existing Air Force bases were examined relative to the cumulative total peak deflection-performance concept, and it was concluded that, though discrepancies where noted between results of the two studies, the idea of a threshold cumulative deflection to pavement deterioration was justified.

The use of this concept for overlay analysis is based on the fact that the addition of an overlay increases the cumulative deflection at which a given pavement condition (e.g., PSI level) occurs and it decreases the rate at which the critical cumulative deflection is approached (Figure 12).

One of the most important links to successful use of this approach is the ability to predict the maximum or peak deflection for any aircraft operating on any pavement structure. The approach proposed by the authors is to utilize transfer-function theory for this procedure. It is important to note that a transfer function is a measure of the relevant pavement parameters, and thus this concept precludes the necessity of assuming a model to characterize the response behavior of a pavement. Successful predictions of deflection basins for aircraft have already been made by this theory (10).

Although the theoretical concepts of the transfer function are beyond the scope of this report, the significant fact is that, given a transfer function of any prime mover on a given pavement system, the signature (peak deflection) can be predicted for any other wheel assembly. It is significant to note that for this procedure it is recommended that a standard prime mover be established to measure and compile function data on any specific pavement structure evaluated.

Finally, a key contribution made by the researchers deals with an initial attempt to measure and define a parameter that would be independent of the magnitude and mode of loading and that would be representative of the characteristics of the pavement system. The value selected to represent this condition was S, a spring constant used to model the entire pavement system. The pavement system reaction S, defined according to energy and work principles, is the ratio of the work done in deflecting the pavement to the value of the energy basin

Figure 11. Predicted effect of surface course thickness on condition of pavement as a function of its cumulative total deflection.

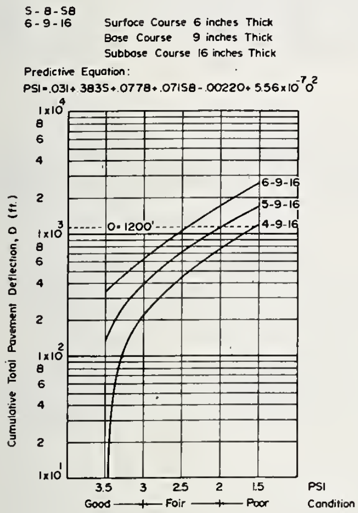


Figure 12. Effect of surface course thickness on service life of highway pavement.

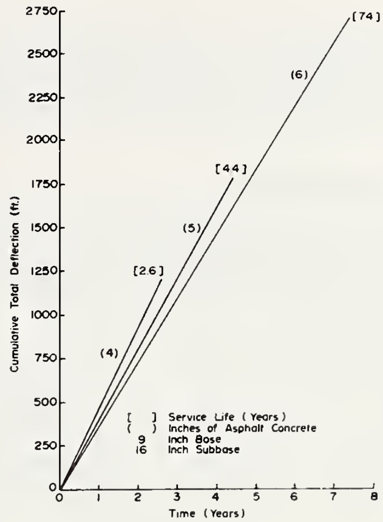


Figure 13. Pavement system reaction coefficient.

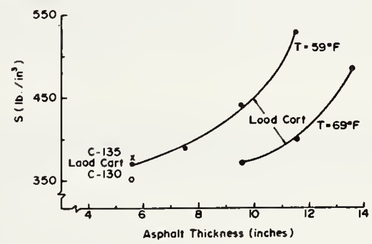
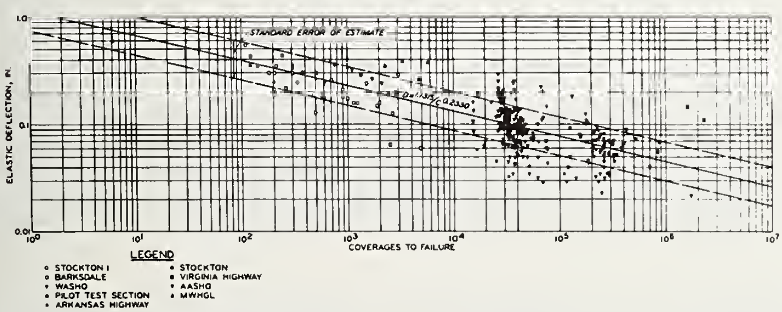


Figure 14. Deflection versus coverage from combined highway and airfield data.



$$S = \sum_{i=1}^{m} \frac{W_i \, D_i}{\int_{x_1}^{x_2} \int_{y_1}^{y_2} D^2 \, (x,y) \, dx \, dy}$$

where W_1 is the weight supported by the i^{th} wheel of an n wheel prime mover, d_i is deflection of the pavement under the i^{th} wheel and $D^2(x,y)$ is the square of the deflection D at point (x,y) of the deflection basin. This equation was solved for several gear loads and pavement sections, and the results are shown in Figure 13. Although some deviation is noted among the C-130, C-135, and load cart S values for the 5.5 inch AC section, the results do not appear to differ greatly.

Non-Destructive Testing

In recent years considerable research has been conducted in the area of non-destructive testing of airfield and highway pavement systems. In general this research may be grouped into three major categories depending on the purpose of the measurements.

Determination of In-Situ Modulus - The use of non-destructive techniques for evaluating in-situ layer moduli has received increased study along with the development of rational or fundamental design theories for flexible pavements. It is important to recognize that the ultimate use of non-destructive testing for this purpose implies that a workable layered theory solution is available. One agency using this type of approach is the Shell Oil Laboratory, which measures dynamic modulus of the layers from dynamic vibratory tests (20, 31). However, most agencies conducting research currently consider this approach only a research tool rather than an acceptable design methodology (68, 109, 110).

The type of dynamic test commonly used for this procedure is called the wave velocity method. The tests measure the response of the pavement to a sinusoidally applied load. Both heavy mechanical vibrators of frequency range 6-80 Hz and electrodynamic vibrators of 20-3,000 cps may be used. By placing transducers at various distances away from the vibrator source, wave lengths being propagated in each layer may be determined. The wave velocity V may be determined by

$$V = f\lambda$$

where f is the frequency and λ represents the wavelength in feet. At any location, several velocities will be measured and assigned to a layer in the pavement structure. By knowledge of the wave velocity of each component layer, the shear modulus and subsequent dynamic E modulus can be computed by $G = \rho V^2$ and $E = 2(1+\nu)\,G$. Specifically,

$$E = 2(1+\nu) \; \rho V^2$$

where ν is Poisson's ratio and ρ is the mass density of the material.

Several attempts have been made to determine the E_i from wave-velocity measurements and then use these values to compare predicted deflection basins to measured values. In general, there are two conclusions that are similar throughout any such investigation. First that the maximum deflections are in relative agreement, and second, that the shape of the deflection basin differs. The probable reasons for the latter are primarily the stress dependency of unbound materials coupled with the restrictions imposed by multi-layered elastic theory to use homogeneous and isotropic layer material properties. In layered theory, non-linear behavior may be indirectly accounted for by stress-iterative solutions; however, this procedure can account for stress dependency only in the vertical direction. To consider radial (horizontal) stress dependency probably requires the use of various finite element procedures to achieve a more complete correlation of predicted to measured deflection basin responses. (54, 68, 109, 120).

Deflection-Extrapolation Procedures - The USACE has suggested use of vibratory load-deflection measurements as a design tool in pavement design, evaluation, and/or potential airfield overlay by (53, 66). This method makes use of an extensive study of elastic deflection-repetition to failure relationship established by the USACE for both highway and airfield pavements (65). This relationship is shown in Figure 14. Of particular significance is the extremely large scatter associated with deflection as a critical design parameter for any desired repetition level.

By using the limiting deflection relationship shown in this figure, a design deflection can be chosen to correspond to any desired coverage level. Using the results of vibratory deflection-load tests on the existing pavement makes it possible to extrapolate these results to the design deflection and obtain the allowable single wheel load for that deflection (Figure 15).

Recent studies conducted by the USACE have shown that this procedure does not result in very favorable comparisons with existing procedures for evaluation (53). As a result, this analysis has been eliminated from future non-destructive studies. Because the deflection coverage relationship shown in Figure 14 is applicable only to flexible pavements, the concept was not applicable to rigid pavement types.

Pavement Stiffness Concept - A promising non-destructive technique that is applicable to both rigid and flexible pavement evaluation and ultimately overlay analysis is the pavement-stiffness concept developed by the USACE (51, 53). As defined by the USACE, stiffness is a measure of the entire pavement structure and represents the ratio of applied dynamic force to the elastic deflection determined at the surface of the structure.

Using vibratory test results conducted on several airfield pavement structures, comparative relationships between measured stiffness and allowable assembly load were determined for several aircraft. Figure 16 shows this data relation for rigid pavements and the C-5A and C-130 aircraft. Based on these results, various non-destructive evaluation curves have been developed for various aircraft and pavement types (Figure 17). As noted, the stiffness-load curves are dependent on various types of traffic categories. As there is a definitive relation of these categories to number of coverages (repetitions) to failure, it is possible to redefine such curves for various failure repetition levels.

The utility of such an approach to overlay analysis appears to be promising. The research required for such an implementable procedure is to ascertain the general increase in stiffness for various additional (overlay) thicknesses. One final consideration of this method is to recognize the similarity between it and the S value calculated from work and energy principles previously defined in the energy-transfer function approach, and to current test methods, such as the modulus of sub-grade reaction, k, and other common plate load test parameters.

Layered-Elastic Design Approaches

Although several multi-layered elastic approaches are available for flexible pavement analysis (118, 127), one procedure is described that is specifically adaptable to airfield overlay and evaluation analysis of both rigid and flexible pavements. This method is based on the concept suggested by Vallerga (120, 133, 135).

Figure 18 outlines the various steps in the complete evaluation procedure. Although the detailed factors of each step may be found in the appropriate references, a general discussion of each step is given below.

Step 1. Condition Survey - The objective of this phase is to have experienced engineers carefully examine the airfield pavement areas to determine the condition of the existing pavements in terms of the nature and extent of cracking, patching, rutting, or any other observable sign of structural distress. This information is initially plotted on a large scale plan view of the airfield facilities and is used to help in the sampling and field test phase as well as establishing the performance-related failure criteria.

Step 2. Deflection Measurements - The purpose of this phase is to determine pavement deflections under known loading conditions and to differentiate areas of statistically significantly different deflection responses. It is important to note that any one of several existing deflection devices can be used (e.g., Benkelman beam, road rater, Dynaflect, etc.)

Step 3. Drilling and Sampling - Using the results of steps 1 and 2, a sampling program is conducted to obtain pavement cores, layer samples, and general information regarding layer thickness, water table conditions etc.

Step 4. Laboratory Testing - Material tests are conducted over the general range of temperature and moisture considered to prevail at the site. Since layered theory is used, the tests needed are modulus tests for material response. In addition laboratory fatigue tests for AC mixtures as well as MR tests for PCC are conducted to define the laboratory failure conditions.

Step 5. Traffic - This portion of the study deals with a detailed examination of the past and anticipated future aircraft traffic projections for each specific pavement route (e.g., taxiway). The use of equating various aircraft types with equivalent damage of a standard aircraft is suggested.

Step 6. Performance-Related Criteria - This important step is to establish criteria related to

Figure 15. Typical vibratory data illustrating deflection-extrapolation technique.

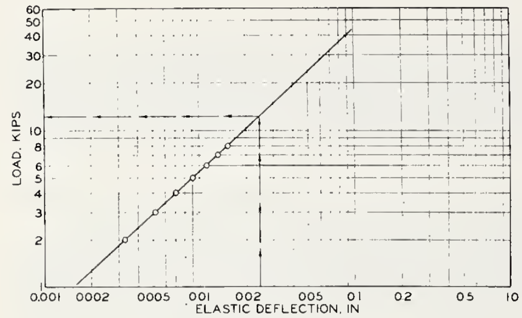


Figure 16. Stiffness-load correlation data for rigid pavement.

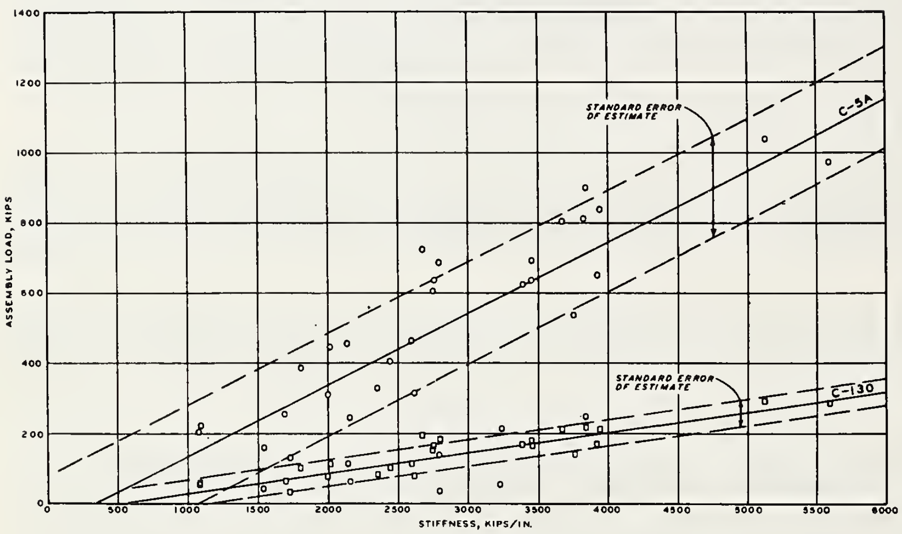
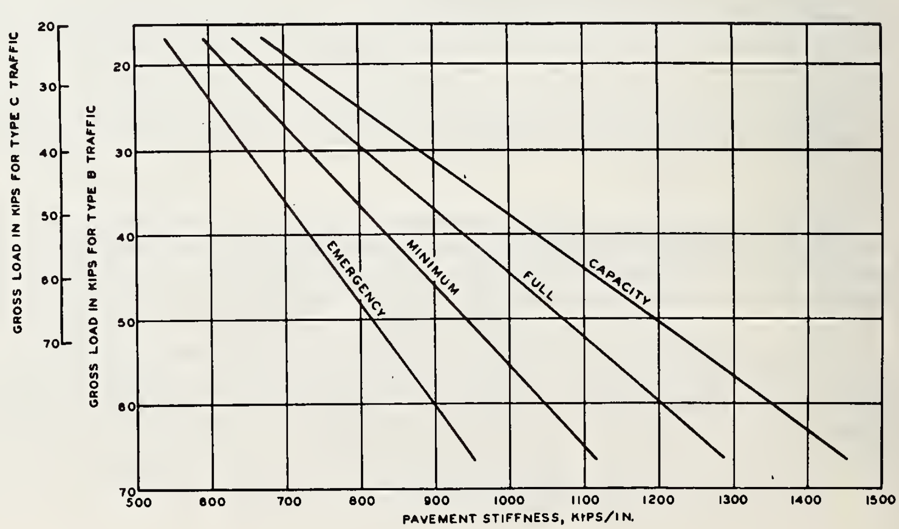


Figure 17. Nondestructive evaluation curves for flexible pavement, single wheel load, 100 sq in.



performance of the various pavements from information obtained from steps 1 to 5. In general, the procedure uses the results of laboratory properties measured in step 4 as initial layered input. Using the results of known deflections found in step 2, adjustments in laboratory E values are made until predicted (theoretical) deflections agree with the measured deflections. Once this is accomplished the critical performance parameters for the aircraft in question are determined from layered theory and related to acceptable and not acceptable performance areas found in step 1 as well as the laboratory-developed failure criteria of step 4.

Step 7. Load Carrying Capacity - From the results of step 6 either the load carrying capacity of the existing pavements or projected overlay requirements for future traffic or a given design life can be readily made.

General Summary of Current Research

Although only a few major research areas have been discussed, a general summary of the major trends in airfield design and overlay analysis are stated in terms of the three factors deemed critical to the pavement analysis: (a) theory, (b) materials characterization, and (c) failure criteria.

Theory - Most rigid pavement designs have been predicated on some modifications of Westergaard's theory for stress evaluation. However in the past several years, the use of multi-layered elastic theory (or finite element type approaches) have been used not only for flexible type construction but also for thick (full-depth) asphalt pavements (127), PCC airfield pavements (98, 104), and CRCP pavements (83, 84).

Materials Characterization - In general, the author believes that most research in this area is focused on developing representative dynamic modulus values of various pavement materials that can be effectively introduced into layered theory to accurately predict critical design parameters. Much work has gone into both laboratory testing and in-situ moduli determination. Although the results of comparative studies of predicted to observed results is very encouraging, a major factor that must be considered is the stress dependency of most pavement materials.

Failure Criteria - Priority research emphasis must be given to establishment of what constitutes failure (whether structural or functional) in airfield pavements, what the critical parameters are, and what level of magnitude can be associated with each parameter before failure occurs. There appear to be two distinct research directions being pursued relative to failure criteria in airfield work. One such direction is to use limiting elastic deflections and the other is to define limiting values of stress or strain at certain critical locations. The use of limiting pavement deflection has several obvious advantages as a failure criteria. However, this parameter by itself cannot be viewed as the sole criterion of performance or failure.

The obvious general relationship between deflection and coverages to failure for pavements is recognized as a well established principle. However, Figure 14 and the large degree of scatter indicate that other factors besides deflection must be responsible for failure mechanisms. Even in the interpretation of what deflection measures,this criterion is sometimes viewed by some agencies as an indicator of fatigue cracking while others interpret this to be related to irrecoverable deformations of the pavement system.

In addition, past research has undeniably confirmed that the limiting deflection of a pavement structure is also a function of the type of base material composing the pavement system. For pavements comprised of thick asphalt layers, this deflection is a function of thickness. Figure 19 shown such a deflection relation for various subbase and base materials obtained from field performance studies.

Recent research studies dealing with overlay analysis in France and South Africa have also shown that deflection in itself may not be the sole indicator of failure (74, 48). For example, Leger has stated:

> "The value of the deflection is a convenient though not exhaustive means of judging the quality of a pavement...this leads to the assumption that deflection is a valid criterion of longevity in the case of pavements which deteriorate mainly as a result of inadequacy of bearing capacity of the underlying soil.
>
> In the case where the criterion of deflection must serve to evaluate the thickness of an overlay made with a given material...it is worthwhile complementing the measure of deflection by a measurement of the radius of curvature of the deflection basin.... Consequently, by

measuring the R_c it is possible to verify that the deformation of horizontal tensile stress at the first interface remains below an admissable limit."

Thus, the French have clearly adopted the concept that deformation is indicated by deflection while curvature measurements relate to fatigue cracking of bound layers. This philosophy is also similar to the overlay concepts proposed by the South Africans (48).

The energy concept noted in this report is a different viewpoint of the limiting deflection analysis and as such suffers from the same difficulties encountered with the deflection criteria. However, it is recognized that the use of the transfer function approach appears promising and should be pursued further.

The other type of failure criterion involves the selection of critical parameters to define failure in a pavement system. In flexible pavement analysis, two recognized modes of failure are permanent deformation of the pavement and repeated load fracture of the system. Critical parameters commonly used are the vertical strain on the subgrade and tensile strain at the bottom of the bound (stabilized) layer or layers. In rigid pavement analysis, it is widely known that fracture initiating from the bottom of the slab has been defined as failure. However, a very recent and important concept recognized for heavy wide bodied aircraft operating on weak subgrades acknowledges that because of the extremely large loaded gear areas, pavement failures in rigid pavements may also occur due to excessive permanent deformation of the subgrade.

When this fact is viewed along with the growing use of multi-layered theory as an analytical tool for all pavement construction, there appears to be significant justification to pursue research leading to a universal design methodology relying on deformation and fracture modes of distress that can be applied to any type of pavement construction, environment, and load conditions.

THE CONCEPT OF REMAINING DAMAGE

The remaining damage concept as a workable pavement design method for overlays was introduced by McCullough (84). It is based on the principle that the service life of any pavement is a function of allowable number of load repetitions the pavement system can sustain. For a given thickness t and a given set of design factors, the number of failure repetitions is given by N_{f1}. For any number of load repetitions, n_ρ occuring up to a certain period of time (n_1 less than N_{f1}), a certain damage D_1 is sustained by the pavement. Because the total damage at failure is equal to one, the remaining damage D_R, is

$$D_R = 1 - D_1 = 1 - \frac{n_1}{N_{f1}}$$

If additional thickness is placed on the system t_o, the allowable failure repetitions for a pavement thickness ($t+t_o$) changes from N_{f1} to N_{f2}. The overlay design is based on the concept that the future damage due to Δn repetitions beyond n_1 ($n_2=n_1 + \Delta n$) must have as a maximum limit of damage the D_R. Therefore,

$$D_R + \frac{N_{f1} - n_1}{N_{f1}} = \frac{\Delta n}{N_{f2}}$$

Knowing the D_R value at the time of overlay and selecting any n_2 value for the design, t_o may be found by the design method used so that $t+t_o$ yields N_{f2} for the same design factors used. It is to be noted that this concept is a valid consequence if the design procedure acknowledges that a direct functional relationship exists between repetition and design thickness.

Because different distress mechanisms (deformation and cracking) may occur in a pavement system, it is logical to assume that damage occurs in two forms. However, the rate of damage accumulation may not necessarily be the same for these different mechanisms. It should be recognized in future research studies that the remaining damage due to cracking, D_{RC}, may not equal the remaining damage due to deformation, D_{Rd}, after n_1 repetitions.

Finally, the applicability of this theory for pavements approacing failure conditions is not well defined. As D_R approaches zero

$$\lim_{D_R \to 0} N_{f2} = \infty$$

Presently there exist two theories on how N_{f2} approaches this limiting value. As an example, these two views may be better explained by using cracking failure criteria for both asphalt concrete and portland cement concrete materials. For both materials, criteria exist that define either

Figure 18. Pavement evaluation method.

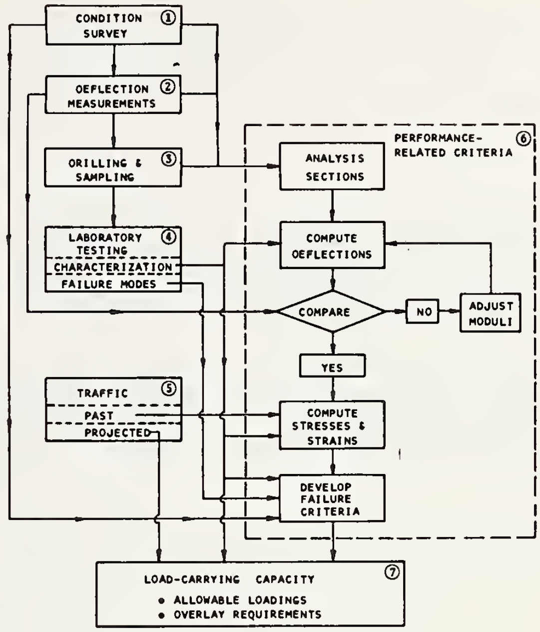


Figure 19. Summary of relations between deflection and critical life.

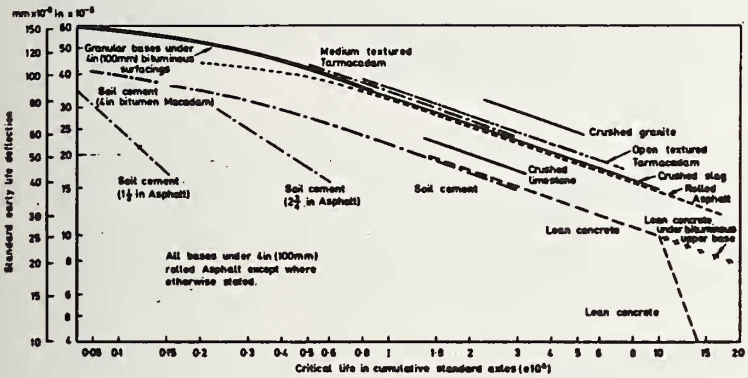
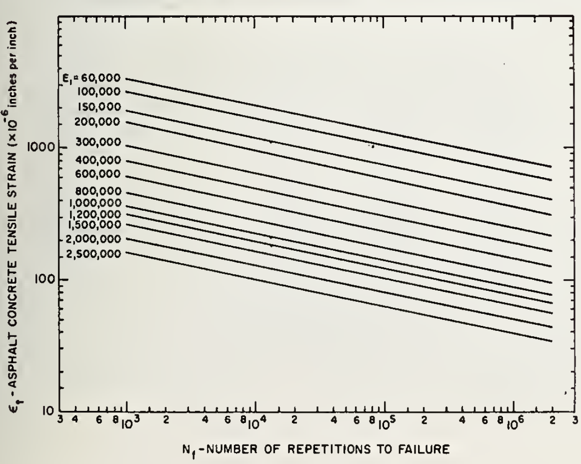


Figure 20. Example of continuous types of AC fatigue criteria.



(a) an endurance limit of stress or strain or (b) a continuous functional relationship between stress or strain and repetitions to failure.

For asphalt concrete fatigue, an example of a continuous tensile strain criterion is shown in Figure 20. This criterion:

$$N_f = K_f \left(\frac{1}{e}\right)^c$$

is valid for all ranges of tensile strain, e. An example of a similar criterion, but one that uses the concept of an endurance limit (e = 70 microinches per inch) is shown in Figure 21. This criterion is of the same form as the continuous type except N_f equals infinity for e less than 70 microinches per inch.

For rigid pavements, the concept of an endurance limit is widely recognized and usually defined as a tensile stress to strength ratio of 0.50 or less. However, fatigue type relationships similar to that for asphalt concrete, have been developed and used in highway and airfield pavement analysis (<u>84</u>). One such example used is the form:

$$N_f = K_r \frac{MR}{\sigma}^{cr}$$

For these two concepts it can be shown that:

$$\lim_{D_R \to 0} t(h) = \infty \qquad \text{(continuous)}$$

$$\lim_{D_R \to 0} t(h) = 0<x<\infty \quad \text{(endurance limit)}$$

This fact has significant design implications in applying this theory for overlay purposes. If a continuous criterion is assumed to govern a pavement that is to be overlaid with a very small D_R value, an overlay thickness approaching infinity would be the solution satisfying this criterion. However, if an endurance limit is used, a finite thickness of overlay would result. It would appear that further research is required to investigate this concept.

Review of Current Design Procedures

None of the current design agencies or procedures presently consider the remaining damage of the pavement as a part of the overlay process. This is probably because the general philosophy in airfield pavement design (new and overlay) is based on a critical future aircraft. In addition, almost all design procedures reviewed (USAF, CDOT, USN, FAA, PCA partially) indirectly account for load repetition level by assuming the design thickness represents a certain level of traffic operations deemed to be at or near capacity for the pavement life.

If remaining damage concepts are to be applied to airfield overlay analysis, a workable and accepted procedure must be introduced to equate the damage effects of the various aircraft types (particularly for civil airfields) to a common or standard aircraft. At present, only two agencies (TAI and PCA) have a working procedure that accounts for the effects of mixed aircraft traffic. The TAI method is based directly on the use of theoretically developed aircraft equivalent damage factors for both deformation and fatigue distress. Although the capability exists within the method to calculate predicted damage for future designs, the specific overlay procedure used is based on an empirical approach that uses the conversion factors for material type and condition previously discussed.

Although the general design procedure recommended by PCA utilizes the critical aircraft concept, an alternate approach based on fatigue concepts for mixed aircraft traffic types and volumes is presented. This procedure (<u>104</u>) has the potential, like that specified by TAI, to be used in overlay analysis using remaining damage concepts.

Table 11 is an example problem (<u>104</u>) for evaluating the mixed traffic effects on design thickness. The basic procedure utilized is to check the adequacy of the pavement for a given projected traffic mix of a designated design period, slab thickness, and subgrade support value. As noted, the predicted stress of each aircraft j is computed from standard design charts or the PCA computer program (col.2) and a stress ratio is determined ($\frac{\sigma}{MR}$ - col.3). Columns 4-6 account for the predicted number of stress repetitions of each aircraft n_j at the critical (maximum damage) location for the expected movements of each aircraft. This analysis accounts for aircraft wander (laterally) characteristics (LRF-col. 5). By using the failure criterion shown in Table 11 (note

Figure 21. Example of endurance types of AC fatigue criteria.

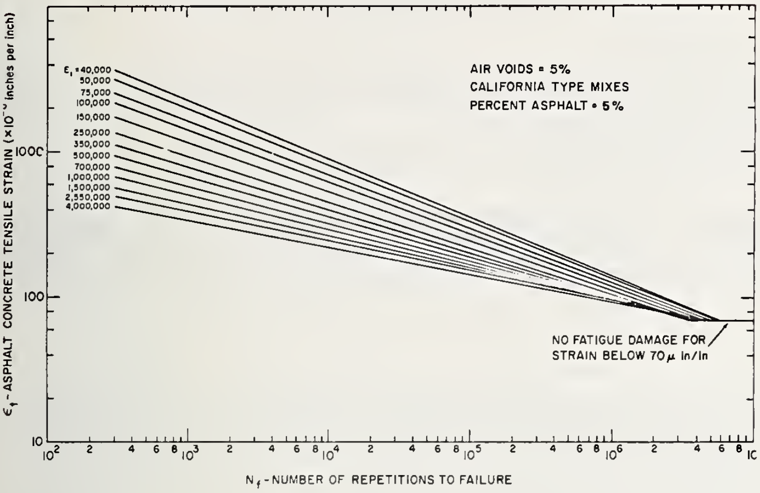


Table 11. Example of remaining damage analysis (from PCA manual).

Pavement Design for Mixed Traffic

Pavement: Taxiway C — Traffic: 2 Million Departures

Slab thickness: 16.0 in. — 90-day modulus of rupture: 650 psi

k value: 300 pci — $^{*}V$ = 18%, M = 1.10, $(1 - V/100)M$ = 0.90

Design modulus of rupture (DMR) = 585 psi

Aircraft (1)	Stress, psi (2)	Stress ratio (3)	Expected number of departures (4)	LRF (5)	Fatigue repetitions (6)	Allowable repetitions (7)	Structural capacity used, percent (8)
Future #4*	354	0.61	1,500	1.33	2,000	24,000	8.3
B-2707**	332	0.57	9,600	0.52	4,990	75,000	6.7
DC-10-X†	330	0.56	32,000	0.57	18,200	100,000	18.2
L1011-X†	324	0.55	15,000	0.57	8,550	130,000	6.6
B747-X†	336	0.57	35,000	0.58	20,300	75,000	27.1
DC-8-63	305	0.52	57,500	0.83	47,700	300,000	15.9
B707	285	0.49	84,000	0.83	69,700	–	0
B747	280	0.48	38,000	0.58	22,000	–	0
DC-10-10	275	0.47	90,000	0.57	51,300	–	0
L1011	270	0.46	24,000	0.57	13,700	–	0
B727	265	0.45	387,000	0.41	158,000	–	0
Other	<270	<0.50	1,227,000	–	–	–	0
						Structural Capacity Used, Total	82.8

Columns 1 and 4 – From traffic projection
Column 2 – From PCA design charts
Column 3 – Stress ÷ DMR
Column 5 – Values from Table A1
Column 6 – Column 4 X Column 5
Column 7 – Values from Table A4
Column 8 – Column 6 ÷ Column 7 X 100

*Projected Future Aircraft No. 4, 1 million lb. gross weight, gear 44x56 in., 4 post (2 tracking).
**12-wheel gear, spacing: 22x44x22 at 44 and 44 in.
†Projected future stretch versions, gross weight assumed 20 percent greater.

Stress Ratios and Allowable Load Repetitions

Stress* ratio	Allowable repetitions	Stress ratio	Allowable repetitions
0.51**	400,000	0.63	14,000
0.52	300,000	0.64	11,000
0.53	240,000	0.65	8,000
0.54	180,000	0.66	6,000
0.55	130,000	0.67	4,500
0.56	100,000	0.68	3,500
0.57	75,000	0.69	2,500
0.58	57,000	0.70	2,000
0.59	42,000	0.71	1,500
0.60	32,000	0.72	1,100
0.61	24,000	0.73	850
0.62	18,000	0.74	650

*Load stress divided by modulus of rupture.
**Unlimited repetitions for stress ratios of 0.50 or less.

that this is of an endurance type) along with the stress ratios of col. 3, the allowable repetitions to failure for each aircraft N_{fj} are obtained and entered in col. 7. Column 8, shows the damage caused by each aircraft D_j over the design time period. Thus,

$$D_j = \frac{n_j}{N_{fj}}$$

and by summing the total damage due to all the aircraft, the damage sustained by the pavement in the design period D_1 (according to previous nomenclature) is

$$D_1 = \sum_j D_j = \sum_j \frac{n_j}{N_{fj}}$$

As can be seen in Table 11, $D_1 = 82.8\%$. Since, by definition, $D_R = 1\text{-}D_1$, the remaining damage for the conditions shown in the example after t years (assumed design period) would be $D_R = 17.2\%$. Even though this procedure can be used to find the D_R value, the use of the remaining damage concept for overlay design is not used by PCA.

Methods of Determining D_R

To determine the remaining damage of an existing pavement requires that the two parameters that the D_R is a function of, namely n_1 and N_{f1} be quantified. There are two ways to determine the n_1 value. They are either by (a) reviewing the previous traffic history of the pavement or (b) direct laboratory measurement to infer the number of repetitions that have been accumulated to date. In addition, there are two possible ways to evaluate the N_{f1} value: (a) direct examination of design curves and (b) laboratory tests. Subsequently, there are four combinations or procedures that may be used to determine the remaining damage:

$$\text{Procedure 1: } D_R = \frac{\text{Previous Traffic History}}{\text{Design Curves}}$$

$$\text{Procedure 2: } D_R = \frac{\text{Previous Traffic History}}{\text{Laboratory Tests}}$$

$$\text{Procedure 3: } D_R = \frac{\text{Laboratory Tests}}{\text{Laboratory Tests}}$$

$$\text{Procedure 4: } D_R = \frac{\text{Laboratory Tests}}{\text{Design Curves}}$$

In developing these procedures it should be recognized that certain characteristics of any individual study may eliminate one or more of these procedures. Recalling that the D_R value may be different for permanent deformation and fatigue cracking, it is difficult to determine the N_f value (D_{Rd}) for deformation from laboratory tests because of the lack of a current available rational or fundamental approach to determine the deformation. In contrast, adequate theoretical concepts and laboratory evaluation procedures are available to determine the N_f value associated with fatigue cracking for both asphalt concrete and PCC. The D_R values may be obtained by using the following procedures:

Procedure 1 - In concept, a careful and detailed study of the previous traffic history must be made to determine the equivalent number of standard vehicle or aircraft repetitions that have occurred from the time the pavement was constructed until the time of overlay. This value is the n_1 factor. If the critical design factors (e.g. thickness t or h, subgrade support CBR, M_R, S) are known, the N_{f1} value for the existing pavement structure can be determined in terms of the allowable number of repetitions of the same equivalent or standard load used to express n_1.

Procedure 2 - The n_1 value is obtained exactly the same way as noted in procedure 1. However, it is assumed that laboratory tests can be used in conjunction with theory to determine the N_f value. For simplicity and reasons previously noted, it is assumed that fatigue cracking is the governing damage for this example description.

Asphalt Concrete Pavement - Step 1: Determine from lab tests and/or assume layered material response E_i, v_i.

Step 2: Determine from laboratory tests the fatigue relationship e versus N_f from beam specimens removed from an untraveled portion of the pavement.

Step 3: For the input values found in step 1 and the standard load conditions used to define n_1, calculate the critical tensile strain of the asphalt bound layer by layered theory.

Step 4: Using the strain from step 3 and the failure relationship of step 2, N_f may be calculated.

Portland Cement Concrete Pavement - Step 1: Determine MR from specimens removed from untraveled portion of pavement and measure or estimate existing subgrade reaction k.

Step 2: Determine allowable $\frac{MR}{\sigma}$ versus N_f relationship from fatigue tests on samples obtained from untraveled pavement portion. An alternate approach is to assume this relationship (Table 11).

Step 3: For the input values found in step 1 and the standard load conditions used to define n, calculate the critical flexural stress in the PCC from theory.

Step 4: Using the stress from step 3 and MR value from step 1, calculate the $\frac{MR}{\sigma}$ value and then determine N_f from step 2.

Procedure 3 - This procedure is based directly on the concepts of remaining damage theory. Figure 22 illustrates generalized lateral distribution patterns of traffic on airfield taxiways and runways and highway pavements. In each of these diagrams, point A refers to an area that is generally untrafficked and as such represents a pavement portion that has not been damaged by load. Point B is typical of the location where maximum damage to date is assumed to have occurred.

Shown below is a typical N_f versus stress (PCC) or strain (AC) diagram. In concept, if samples are tested from A, an allowable $E-N_f$ or $\sigma-N_f$ curve could be developed and is illustrated by A on the diagram. From remaining damage concepts, it would appear logical to assume that, if samples were removed from B areas and tested, the N_f curve would no longer correspond to the A curve because this material has been damaged to some degree by n_1. In other words the number of repetitions to failure at any given σ or e level should be $N_{f1} - n_1$ or N^1. From theory, this can be seen to be a viable approach to measuring the D_R value directly from tests procedures as:

$$D_R = \frac{N^1}{N_{f1}}$$

This procedure affords the possibility of conducting well designed statistical experiments that could be conducted only at a given stress or strain level in lieu of defining the N_f values for a range in stress or strain.

Procedure 4 - This procedure, although a logical combination of the methods stated previously, is not really recommended. In order to obtain the n_1 values from laboratory tests as explained in Procedure 3, one also must determine the N_{f1} value directly from these tests. Therefore the use of design curves as recommended in Procedure 1 is redundant of the values obtained from Procedure 3.

Examples of Actual Airfield Performance Studies

In this analysis, the concepts used in Procedures 1 and 2 were evaluated. A study was conducted that included the use of remaining damage theory for an existing airfield taxiway pavement at Baltimore Friendship International Airport (136).

The taxiway section (TW A-5) is a full-depth AC pavement constructed in 1951 to a thickness of 10.6 inches over a granular subgrade. In 1963, a 2.o inch overlay was placed in the section. The reason for the overlay was to strengthen the section to accommodate the jet aircraft. the pavement had very little if any observable distress at the time of overlay. After over 20 years of service, the now 12.6 inch section developed extensive longitudinal cracking in the main gear wheelpaths.

Based on an extensive analysis of the traffic history and a crack survey conducted on the taxiway, the number of equivalent cumulative DC-8-63F strain repetitions to failure was calculated as $17{,}016(n_2)$. The number of repetitions accumulated to the time of the 1963 overlay was $4{,}200(n_1)$.

Procedure 1 Analysis - Using the information noted gives the damage equation for a failure condition of TWA-5 as

$$D_1 + D_2 = D_t = 1$$

Figure 22. General procedure of determining remaining damage by lab tests.

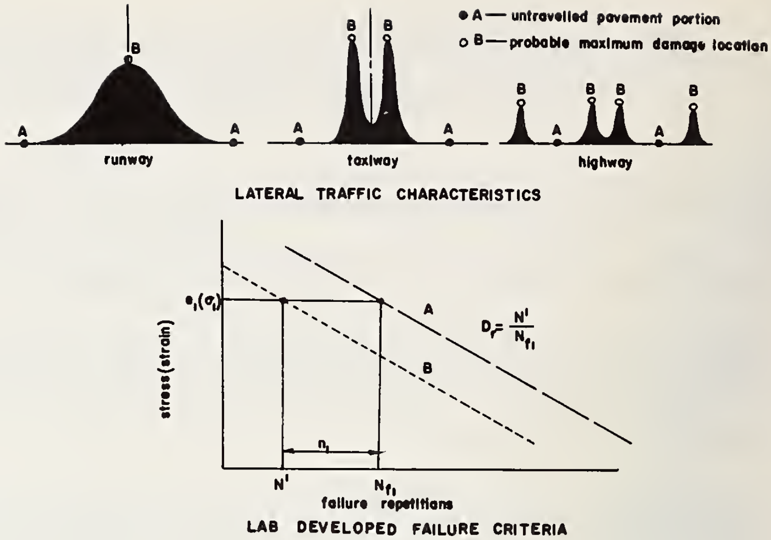


Table 12. Input used for remaining damage analysis from lab fatigue curves: TS A-5.

Method of E_1 Evaluation	A.C. Modulus E_1	Effective Temperature	Fatigue Equation	Tensile Strain for T_A T_A=10.6"	Tensile Strain for T_A T_A=12.6"	Predicted Failure Repetitions T_A=10.6"	Predicted Failure Repetitions T_A=12.6"
Lab Tests $E_{s(\sigma)}$	512,000 psi	55°F	$N_f=2.132x10^{-8}\left(\frac{1}{\varepsilon}\right)^{3.468}$	$380x10^{-6}$"/"	304	15,488	33,580
Lab Tests $\|E^*\|$	890,000 psi	55°F	$N_f=2.571x10^{-15}\left(\frac{1}{\varepsilon}\right)^{5.15}$	263	211	7,036	21,880
TAI Predicted $\|E^*\|$	820,000 psi	55°F	$N_f=3.934x10^{-15}\left(\frac{1}{\varepsilon}\right)^{5.15}$	276	223	8,397	25,180

Table 13. Summary of predicted repetitions to failure for TW A-5 by remaining damage analysis.

Method of E_1 Evaluation	Repetitions to Failure from Fatigue Curve N_{f1}	Repetitions to Failure from Fatigue Curve N_{f2}	Damage Expressions: Before Overlay $d_1 = \frac{n_1}{N_{f1}}$ + After Overlay Failure $d_2 = \frac{\Delta n}{N_{f2}}$ $d_t = 1$	Predicted Repetitions $n_2 = N_{fp}$	Observed N_{oo}	Observed N_{fo}	Factor of Safety F.S. = $\frac{N_{oo}}{N_{fp}}$	Factor of Safety F.S. = $\frac{N_{fo}}{N_{fp}}$
Lab Tests $E_{s(\sigma)}$	15,488	33,580	$\frac{4200}{15,488} + \frac{(n_2 - 4200)}{33,580} = 1$	28,673	13,676	17,016	0.48	0.59
Lab Tests $\|E^*\|$	7,036	21,880	$\frac{4200}{7,036} + \frac{(n_2 - 4200)}{21,880} = 1$	13,020	"	"	1.05	1.31
TAI Predicted $\|E^*\|$	8,397	25,180	$\frac{4200}{8,397} + \frac{(n_2 - 4200)}{25,180} = 1$	16,785	"	"	0.81	1.01

or

$$\frac{n1}{N_{f1}} + \frac{n2-n1}{N_{f2}} = 1.0$$

From the results of the traffic study, n_1 = 4,200 and (n_2-n_1) = 17,016 - 4,200 = 12,016. Because failure of the section had occurred, n_2 represents the total number of repetitions to failure of the section.

To determine the appropriate N_{f1} and N_{f2} values from existing design curves, use was made of the allowable fatigue curves shown in TAI MS-11 (127) for a measured M_R = 25,500 psi. These curves result in N_{f1} = 9,400 for T_A = 10.6 inches and an N_{f2} = 23,000 for a T_A 12.6 inches.

Using the general damage equation noted, along with the nl, N_{f1}, and N_{f2} values stated, n_2 was determined to be 16,924 repetitions to failure for the section. This result obviously is in excellent agreement with the actual n_2 value of 17,016 obtained from the traffic analysis.

Procedure 2 Analysis - The N_{f1} and N_{f2} values were computed from laboratory fatigue test results. Using this method gives the general damage equation at failure for TW A-5 as

$$\frac{n_1}{K\left(\frac{1}{e_{t1}}\right)^c} + \frac{(n_2 - n_1)}{K\left(\frac{1}{e_{t2}}\right)^c} = 1$$

In this equation e_{t1} refers to the tensile strain for the 10.6-inch thickness while e_{t2} refers to the 12.6-inch pavement (initial plus overlay). To obtain a general assessment of this approach, calculations were made for fatigue equations defined by three types of AC modulus-temperature relationships. The fatigue equations were defined at an effective temperature of 55 F. The three modulus relationships used to compute strains determined from constant stress fatigue tests on A-5 specimens were (a) the flexural stiffness, (b) dynamic modulus, and (c) dynamic modulus predicted by the TAI equation.

Table 12 gives the basic input for each method of modulus evaluation and subsequent fatigue equation developed. In addition, the computed tensile strains from layered theory for both the 10.6 and 12.6-inch sections are shown along with the predicted repetitions to failure (N_{f1} is equivalent to repetitions under 10.6 inches and N_{f2} is for 12.6 inches).

Table 13 is a summary of the calculations of the remaining damage analysis. The predicted failure repetitions, n_2, obtained from each method with the general damage is noted. The N_{oo} value represents the observed number of strain repetitions to initial cracking while the N_{fo} value is the observed failure repetitions. Computed factors of safety are shown for each method.

The two methods using dynamic modulus values yield close and conservative estimates of the failure repetitions at an effective temperature of 55 F. The method using flexural stiffness gives an unconservative estimate. Although the implications of each method used are discussed in great detail in the report, it is hoped that these examples using remaining damage principles do afford some insight into the merits and validity of this philosophy. This is particularly significant when it was found that conventional overlay design techniques, which do not account for remaining damage, gave unconservative estimates of the failure repetitions.

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REFLECTION CRACKING

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At the 1932 Annual Meeting of the Highway Research Board, a symposium on Resurfacing Portland Cement Concrete Pavements was held. Forty-one years later we held a Workshop on Pavement Rehabilitation and one of the subjects to be covered is reflection cracking. Obviously, the problem has not been solved, and there may even be some doubt that substantial progress has been made.

In this Workshop there are two principal types of distress being considered. The first concerns the structural adequacy of a pavement and the layers or action needed to establish the road in a condition to carry expected traffic. The second is reflection cracking or the propagation of cracks and joints in the existing surface of portland cement concrete (PCC) or asphalt concrete through the resurfacing layer.

The problem is a serious one and many remedies have been tried over the years to eliminate such cracking or at least to slow its progression. Reflection cracking develops from movement of the slabs under the overlay. It can be caused by differential vertical movements due to wheel loads and the lack of load transfer at the crack or by temperature of moisture changes in the base layer, causing a horizontal movement at the crack. There is no rational, proven method whereby a design can be made with full assurance that these cracks will not reappear. Most of the concern for reflection cracking in the past has been with the cracks that are propagated through an asphalt concrete layer placed over PCC. Reflection cracking, however, can and does exist on flexible pavements where the asphalt layer is over an untreated base, a cement-treated base, or a bituminous-treated base.

To understand the problem more thoroughly, perhaps we should review some of its historical aspects. At the 1932 Annual Meeting Gray and Martin (1) reported that reflection cracking could be controlled with 3-inch penetration macadam or with a 3-inch plant mix surfacing over badly cracked PCC surfaces. For normally cracked surfaces, however, 2 inches of plant mix surfacing would be satisfactory. In 1973, Hensley (33) reported similar success in retarding reflection cracking in Arkansas by the use of 3 inches of open-graded base course covered by 2-1/2 to 3 inches of dense-graded asphalt concrete.

Today there is considerable talk on the use of PCC overlays not only to solve the reflection cracking problem but to solve structural problems or to improve skid resistance. Do you think this is new? At the 1932 meeting, Flemming (2) reported on a resurfacing project that placed 10 feet of PCC in the center of Union Street in Schenectady, New York, in 1909. This PCC resurfacing was 1 to 1-1/2 inches on the edges and 3 inches in the center. Of more than passing interest was the fact that the surface was grooved to improve traction. California placed 200 miles of 4-sack PCC between 1912 and 1917. Resurfacing of these roads commenced in 1919 and continued through 1930 with thicknesses of 4 to 6-1/2 inches of PCC. From this work came the conclusion that a minimum of 4 inches of PCC overlay was needed to carry the amount of traffic of that day and 5 inches was needed over badly cracked PCC. Also of interest was the conclusion that special measures to prevent bond were not justified. This was determined by some pavements that had been overlaid with seal coat bituminous layers.

In 1957 the Highway Research Board published an annotated bibliography (3) containing 175 references on rigid overlays and 80 on bituminous overlays. Not all of these, of course, were for the purpose of preventing reflection cracking. Many were for the purpose of increasing the structural strength of the total section.

The first systematic approach to evaluating factors affecting asphalt cement resurfacing over concrete pavements was published in 1955 by Crump and Bone (4). They reported on a project of 25 test sections 1,000 feet long and arrived at two important conclusions:

1. The successive record of various sections showed a definite progressing of reflection cracking in the early years. Different types of cracking develop at different rates, but all types increase with each additional climatic cycle. For instance, by the third year many pavements had cracks over more than 75 percent of the length of the transverse joint. A few reached 95 to 100 percent in 3 or 4 years. Successive surveys show a progressive growth of crack width after each annual temperature cylce rather than an increase in length of cracking.
2. Resurfacing laid in the fall does not usually crack until the second winter, while that laid in the spring or summer almost always begins to crack during the first. After about 2 years the cracking was about equal on both types.

The mechanics of reflection cracking were examined in detail in 1954 by Bone, Crump, and Roggeveen (5). Their paper detailed that, although reflection cracking may result from either vertical or horizontal movement of concrete slabs, horizontal movement appeared to be the predominant factor. This movement, which results entirely from moisture content and temperature variation, was found to vary seasonally from 0.10 to 0.18 inch per 57 feet (the distance between expansion joints in Massachusetts). Because of the roughness of the concrete and natural adhesiveness of the asphalt concrete (AC) overlay, movement is not absorbed through any appreciable length; therefore, the surface immediately adjacent to and directly over the concrete joint must conform to the entire contraction. Unfortunately, at low temperatures where maximum contraction takes place, AC surfacing is in an elastic state. Results indicated that at 30 F, AC surfacing specimens of average quality and 12 inches in length would not tolerate strains in excess of 0.05 inch over a 4-inch gauge length without cracking. These results were verified by field measurements which found that, in nearly every instance where the surfacing had stretched more than 0.05 inch, a crack appeared.

In a study of reflective cracking in Iowa reported by Roberts (6), particular emphasis was placed on width of pavement joints, type of joint filler, and thickness of surfacing. It was found that,over cracks less than 1 inch in width, the percentage of reflection cracking was only slightly affected by the type of joint filler. Pronounced reductions were noted, however, by using an extra 1-1/2 inches of surfacing (increasing from 3 to 4-1/2 inches) and, to a lesser extent, a higher penetration asphalt binder.

In the early 1950's, the use of welded wire reinforcement was tried by several states. California reported in 1956 (31) that some test sections had a significant reduction in the amount of reflective longitudinal cracking through the wire mesh. These results, however, were obtained from sections with approximately 3 inches of surfacing. Sections using welded wire fabric and 4-1/2-inch surfacing failed to produce reflective cracks, indicating that the thickness of surfacing is an important factor in the prevention of cracking.

An additional California project (7), constructed in 1957 with a tapered test section (2 to 5-inch asphalt concrete) using 3- x 6-inch 10/10 welded wire fabric, again illustrated the benefits of increased thickness. In this case, serious corrosion developed in the wire fabric and after 4 years much of the reinforcing effect was lost.

Tons, Bone, and Roggeveen reported in 1961 on the 5-year performance of two test roads in Massachusetts (8). It was found that the best resistance to reflection cracking was provided by continuously welded wire fabric reinforcement (3-x 6-inch, 10/10), which reduced the reflection of transverse cracks to approximately 1/8 of that in the control sections. The use of strip wire fabric reinforcement placed only over cracks resulted in reductions in reflection cracking of up to 2/3. Cracks in the mesh sections were found to be consistently smaller in width.

In 1968 Karanowski (9) summarized the experiences of 20 agencies that had used wire reinforcement in asphalt and found that performance ranged from poor to excellent, with Illinois reporting good performance after 11-1/2 years of service. He concluded that the method of placing the welded wire fabric played an important part in performance of the material.

Another approach to the problem was presented to the Association of Asphalt Paving Technologists (AAPT) in 1963 by Vicelja (10) in which the results of a 4-year study of the effectiveness of expanded wire mesh reinforcement, aluminum foil, wax paper, and stone dust for breaking the bond over existing PCC shrinkage cracks were discussed. Of particular interest is the effectiveness of an 18-inch swath of stonedust in reducing bond and retarding reflective cracking. After several years of observation, the dust bond breaker had been completely successful as compared to the failure of expanded wire mesh and only partial success of wax paper and aluminum foil. It is interesting to note that recent oral reports from Los Angeles County indicate that the bond breakers are still performing satisfactorily after 9 years of service; however, thicker (4-inch) resurfacings are more effective than the thinner (1-1/2-inch) resurfacing layers. Summarizing the effects of bond breaker utilization by 9 agencies, Karanowski reported that most regarded the process as ineffective and only two achieved good to excellent results (9).

One successful method of retarding reflection cracking is that of breaking up the old concrete pavement into smaller segments by using a heavy pneumatic roller of 50+ tons. This method was first reported by Kipp and Preus in 1950 (11), by Velz (12) in 1961, and by Korfhage (13). Karanowski (9) provides summary information which shows that, of the five states reporting, all have had apparent success in retarding reflection cracks by using heavy rollers to break and seat PCC pavement. Results ranged from good to excellent for pavements ranging from 5 to 10 years old.

Another method for reducing reflection cracking is the use of aggregate base cushion course. This method requires the placement of several inches of crushed rock base over the old PCC pavement with a new AC layer placed on top of the base. While successful in many instances, this method does raise the grade a substantial amount. For urban freeways, the amount of grade raise (10 to 12 inches) would probably be intolerable as well as provide some very difficult traffic control problems while construction is in progress. Karanowski (9) reported on the experiences of 8 states, with variable results ranging from poor to very good or excellent. California made a study in 1966 on the effectiveness of flexible base cushion courses over old pavements (34) and found similar results. The study shows that cushion courses of 4 inches developed some reflection cracks while courses of 6 inches or greater did not. One of the problems of using the untreated base is the possibility of base failure because of moisture entrapped under the AC pavement, or because of insufficient thickness of AC pavement over the base material. The development of chicken wire or alligator cracking in such a structure is not usually reflection cracking but a failure of the new structure.

Arizona uses what they call an upside-down design for treated bases in which the cement-treated base is covered with a layer of aggregate base and then topped with AC. This aggregate layer has prevented reflection cracking to a large degree. Cushion courses apparently work in three ways to reduce reflection cracking: they reduce the vertical movement in the AC surface due to the total structural thickness of the mass being placed on the old concrete; they insulate the concrete from temperature and moisture changes so that the total horizontal movement of the old slabs is probably reduced to a level that can be tolerated by the AC pavement; and they distribute concentrated stresses at PCC joints.

Another method of reducing reflection cracking is to overlay the PCC pavement with another layer of PCC. In one of the most recent of many references on this procedure, Martin (14) points out that the most common method of determining concrete resurfacing thicknesses on existing concrete pavement was developed by the Corps of Engineers (15). The most common methods for determining slab thickness of concrete resurfacing on bituminous pavement are those used by the Portland Cement Association (PCA) (16) and the American Association of State Highway Officials (AASHO) (17). Martin also lists 17 overlay projects constructed since 1959 with thicknesses ranging from 6 to 9 inches. The four projects constructed in California have shown no sign of reflection cracking though the oldest of these is now 5 years old.

Illinois analyzed the performance of 89 resurfaced PCC pavements relative to the performance expected from new rigid and flexible pavements. From this analysis a resurfacing design procedure was developed based on a modification of the currently used Illinois rigid pavement design procedure. This work was reported by Elliott (18) in 1971. The Illinois study revealed that the ASSHO present serviceability index (PSI) equation for flexible pavements appeared to be applicable in measuring performance of resurfaced pavements, except that the cracking term should be modified to consider reflection cracking. Therefore, modifications were made to include a cracking term similar to that used in the rigid pavement PSI equation. Formulas developed by Illinois are as follows:

$$PSI = 10.91 - 3.90 \log \overline{RI} - 0.01 \, (C+P)^{1/2} - 1.38 \, \overline{RD}^2 - 0.09 \, (C^1)^{1/2}$$

where $\overline{RI}$ = roughness index measured by Illinois roadmeter
C = a measure of alligator cracking
P = a measure of patching
$\underline{C^1}$ = a measure of reflective cracking
$\overline{RD}$ = a measure of rut depth

Designing to PSI end points of 2.0 and 2.5, and knowing the design traffic load (18 K equivalent over the design period) and the California bearing ration (CBR) of the underlying soil have yielded nomographs to determine the structural number of the road. The required thickness of resurfacing can then be determined by using the thickness of the existing pavement components. If the resurfacing is to be the first for the pavement, the thickness is determined by the equation

$$D_F = \frac{SN_R - 0.26 D_C}{0.40}$$

For second resurfacings the thickness is determined by

$$D_S = \frac{SN_R - (0.25 \, D_F + 0.17 \, D_C)}{0.40}$$

where D_C - thickness of PCC pavement.

The limitations placed on the design system are 3 inches for primary and Interstate roads, 2-1/2 and 1-1/2 inches for roads of lesser traffic or importance. The reason for the minimum is worthy of consideration in that a degree of smoothness is required and Illinois believes that it requires two-course construction to obtain this smoothness.

One of the big reflection crack problems is that caused by the use of cement-treated materials beneath AC. Because the cement treatment is normally a rigid structure, it acts like concrete, expanding and contracting according to temperature and moisture variations. Some studies by California have indicated that various aggregates have different susceptibilities to shrinkage and cracking; however, standards are difficult to specify inasmuch as shrinkage and expansion also vary with the environment as well as the cement content used in the base.

The cracking of cement-treated base largely follows the pattern of concrete. Because it does not break into uniform cracks like those found in sawed concrete contraction joints, cracks in the asphalt concrete have a random appearance. In colder areas where it is necessary to sand the roads, these cracks, as they widen, fill with sand causing the AC to bulge on either side of the crack thereby producing a rough ride on the road. Further, when it is time to resurface such a road, problems arise generally associated with resurfacing concrete pavement. The cracks tend to reflect through the pavement. The reflection may be accelerated by traffic but is largely an expansion-shrinkage phenomenon. Figure 1 shows the end of the Grover Shafter Freeway in Oakland, California. No traffic has been on this road and the three bituminous surfacing layers are evident, with cracks penetrating the first layer (0.2 ft), the second layer (0.15 ft) only partially cracked, and the third layer (0.15 ft) showing no cracking. The same uncracked surface appearance may be found on the freeway where it is carrying traffic on the full 0.5-foot depth of surfacing. This leads one to believe that, with sufficient cover over either PCC or cement-treated base, the reflection cracks will not come through in a very rapid manner. In climates of more severe weather than that found in Oakland, California, the reflection cracking might have progressed through the entire 6-inch depth of pavement. To reinforce this observation, let us consider the 6-inch AC overlay placed on a portion of the Hollywood Freeway in Lost Angeles (8 inches of unreinforced PCC) in 1957. Fifteen years later less that 10 percent of the cracks have reflected through the surface course.

Figure 1.

Reflection cracks not only are developed by cement-treated bases and PCC but also can be developed from AC over untreated base materials. This is particularly true where thermal cracking appears in the higher elevations, but even in milder climates where alligator cracking has been known to reflect through where insufficient cover is placed over the pavement.

With the great increase in multilane road mileage, it is quite obvious that most of the structural problems are in the outer lane, while the inner lanes are still in satisfactory condition and adequate for the traffic that uses them. Currently, much effort, time, and money are being spent to find ways to prevent reflection cracking with a minimum thickness of AC in the truck lanes and thereby save expenditures in resurfacing inner lanes and shoulders. Some of the things that are being tested are

1. Asphalt rejuvenating agent
2. Asphalt emulsion slurry seal
3. Heater remix process
4. Plant mix seal coats
5. Rubberized AC
6. Rubberized slurry seal
7. Rubberized seal coats
8. Hand-poured filling of the cracks prior to overlay
9. Polypropylene fabric

These methods of preventing or reducing reflection cracking are being tried by several states under an FHWA demonstration project. Although it is too early to come to positive conclusions on the best of these methods, early indications would clearly eliminate some of them as a method of preventing reflection cracking, at least under thin AC surfacings.

The use of rejuvenating agents does not appear to prevent reflection cracking. It does, however, soften the surface and provide a crack that surface heals under traffic or at least stays to a minimum width under traffic. In achieving this softness, we run some risk that the surface will bleed and cause reduced skid resistance. This is particularly true when the rejuvenating agent is applied to a mix with ample asphalt and minimum voids.

The use of an asphalt emulsion slurry underneath an overlay would appear to be an excellent way to fill larger cracks and thus minimuze the ability of wide cracks to reflect. To date, the slurry seal has not been effective in preventing such reflection cracking, but most of the trials have been with fairly thin surfacings of 1 to 2 inches.

The heater mixer resurfacing project has proved to be fairly successful, at least in California, in retarding reflection of alligator cracking on old pavements. In this process the pavement is heated, scarified, and remixed to provide a smooth layer for the overlay to be placed on. It has been used successfully in the San Francisco area on freeway ramps. Unfortunately, the use of the heater scarifier has been prohibited in Los Angeles because of pollution problems. Whether this ban will be adopted by other areas remains to be seen.

Open-graded plant mix seals have been used to cover up cracked pavements, and it has been our experience that plant mix seals do not prevent reflection cracking. Eventually the reflection cracks will come through and the open-graded mix will have a tendency to spall.

One of the potential possibilities of retarding reflection cracking is through the use of additives to modify the characteristics of the AC. So far this has been done primarily through use of rubber. Roggeveen and Tons (19) reported in 1956 on the use of

1. Emulsified rubber asphalt -- 5 percent (of weight of final asphalt content) of rubber latex added and mixed with heated aggregate.
2. CRS synthetic rubber -- Powder 100 percent passing a No. 20 mesh sieve. It was cooked into the asphalt at 225 F in the amount of 7 1/2 percent by weight.
3. Natural rubber crumbs -- Crumbs added directly into the pug mill in the amount of approximately 7 1/2 percent by weight of asphalt to the top course and 5-3/4 percent to the binder course.

Roggeveen and Tons concluded that 80 percent of the total transverse cracks reflected through in 4 years and 90 percent after 6 years regardless of whether rubber additives were combined with the asphalt. A similar study by the British Road Research Laboratory (20) also indicated that rubber in asphalt did not reduce cracking. Experience in California (7) has been that while the crack pattern propagation appeared retarded the ultimate performance is no better than the control.

Other types of rubberized applications may be described as a rubberized emulsion that penetrates the pavement, carrying rubber particles with it, which combine with the asphalt. Results to date are inconclusive for this type of material, but it seems to have an effect similar to unrubberized rejuvenating agents in that cracks appear finer and tend to surface heal in warm weather.

Lagrone and Gallaway (20) reported on the use of vulcanized rubber aggregate in an emulsion slurry seal as a stress-relieving layer under AC to alleviate reflection cracks.

They recommend a 1/4-inch layer between the resurfacing and the cracked pavement. This thickness accommodates about 0.20 inch of horizontal base movement. They also suggest the possibility of successfully placing relatively thin (2 to 3 inches) AC blankets over old PCC pavement. Test sections are currently in progress in Texas, North Dakota, and California. Initial results are encouraging but not conclusive.

McDonald (32) reported success in preventing reflection cracking by using a surface seal coat composed of 3/8 x 1/4 inch chips and a binder of 120 to 150 penetration asphalt containing 25 to 30 percent of granulated tread rubber reclaimed from discarded automobile tires. The rubber is ground to pass the No. 16 sieve and to be retained on the No. 25 sieve. This seal coat produces a very flexible surface that will resist both fatigue and reflection cracking. The process is described by Olsen (22).

Crack filling is a recommended practice for all overlays where the cracks are of such nature that they might reflect through the overlay. This would include AC thermal cracking. This process will not prevent reflection cracking but does retard the reflection of large cracks.

For PCC pavements, large cracks should be filled and rocking slabs stabilized (providing they are not broken up with heavy rolling) by pressure filling the voids under the slabs with a lime or cement grout.

Polypropylene fabric is becoming more popular and is being tried by several states (24). While none of the installations has existed long enough to determine the long-term effects of the polypropylene mat, it would appear that it is effective in some cases in retarding cracking. It is being tried both as a full lane coverage material and as a strip material over specific cracks. In California, experience to date indicates that the material is more effective as a full lane width cover material on transverse rather than longitudinal cracks.

Wilson (25) describes successful experiments in controlled reflection cracking. This was accomplished by sawing 3/8 inch wide by 12 to 1-3/4 inch joints in the AC overlay at the existing PCC joints. This controlled cracking procedure has not been widely used and very little reported in the literature.

Roberts (26) reported that experiments in Iowa showed substantial differences in reflection cracking between asphalt mixtures made with 80 and 115 penetration asphalts. At the age of 51 months, sections with the harder asphalt showed 75 percent reflection cracking while the softer asphalt section showed 37 percent. The work done by Hass et al. (27) in Canada would tend to reinforce this use of softer asphalts. Also important is the temperature susceptibility of the asphalts as well as the actual consistency of the material in place after processing through an asphalt plant.

Pretorius, et al. (28) have reported on research to predict shrinkage stresses in pavements containing soil cement bases. Their approach is important, for it represents one of the few attempts to rationalize the cracking of bituminous layers placed on elastic bases. The research used the creep characteristics of all the materials in the pavement structure together with the shrinkage and strength characteristics of the soil cement in an incremental axisymmetric finite-element solution. The authors believe that it is possible to approximate the crack spacing in a soil cement base pavement if the actual distribution of shrinkage strains in the pavement are known.

In an application of fracture mechanics, Ramsamooj (29) developed a theoretical approach to the problem of predicting the rate at which cracks in an underlying layer of pavement will reflect through a bituminous overlay. He considers the initiation and growth of cracks under the combined influence of repeated vehicular loading and changes in temperature. Both the theory of linear elastic fracture mechanics and that of delayed fracture in viscoelastic materials are used to formulate the method of solution. Ramsamooj points out the need for research to verify the method experimentally. Such proof of validity would minimize the need for "trial and error" research on the endless stream of products proposed for eliminating reflection cracking.

McCullough (30), summarized much of the work that has been done on reflection cracking. I wish to acknowledge that many of the references used in this paper were obtained from his excellent report.

Up to this point much of the work on reflection cracking has been of an empirical nature, and a large part of the effort has only been a trial with no real research involved. As determined in the literature review, treatments that worked in one area failed in another. Documentation on the reasons for success or failure is scarce. Follow-up studies to determine life expectancy of treatments have been neglected.

It seems probable that our ability to correct structural deficiencies is exceeding our ability to prevent reflection cracking. Overlays that are of sufficient thickness structurally may have reflection cracks in 1 or 2 years. Vertical movements can be measured with sufficient accuracy by a Benkelman beam or various vibrational devices. The horizontal movement of material under the surface is less easily determined, for it is largely dependent on the environment, which is a less predictable factor. More information is needed to be able to predict maximum movements. It is necessary to learn more about how the cracks propagate and to develop means of retarding the cracks. Finally, better data banks are needed so that the performance of pavements can be documented and research concentrated on the most costly types of distress.

As a result of 41 years' concern with reflection cracking, we have now reached a point where the need for a "rational" design method is recognized. Perhaps from this Workshop a plan can be evolved to systematically attack the problem.

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THE THERMAL CRACKING PROBLEM

AND

PAVEMENT REHABILITATION

W. A. Phang and K. O. Anderson

Non-load-associated, thermally induced transverse cracking of bituminous pavements has received a great deal of attention during the past decade. Considerable effort has been expended toward understanding the basic mechanisms causing such cracking and in developing methods to prevent or retard distress of this type. The problems caused by this low-temperature shrinkage phenomenon are of direct interest to areas that experience severe winter conditions, such as in Canada and the northern United States.

An attempt was made to summarize the progress of many Canadian researchers during the 1960s (1) to be updated as new information became available (2). From the earlier emphasis on defining the severity and extent as well as the cause of the problem, recent attention has been placed on the development of practical engineering solutions. One such monumental work is the comprehensive documentation of work toward designing asphalt pavements against low-temperature shrinkage cracking reported by Haas (3). Much of this work involved the evaluation of existing pavements to assist in the development of design methodologies for new pavements, although it is recognized that the rehabilitation of existing cracked pavements is also a major concern. Hopefully, implementation of principles clarified through this earlier phase of endeavor will lead to improved rehabilitation practices. A brief state of the art concerning thermal cracking is therefore intended to draw attention to some of these major principles. The overlay treatments described herein are not intended to prevent or retard low-temperature, transverse shrinkage cracks from reflecting through into the overlay. Rather, they are designed to cope with differential movement of the underlying pavement structure.

THERMAL FRACTURE MECHANISMS

Regularly spaced, transverse cracks in association, at times, with longitudinal cracks along the centerline and shoulders of paved surfaces, occurring in low-temperature climatic environments, have come to be recognized as thermally induced. The extent of cracking has been expressed by numbers of cracks per mile, average crack spacing in feet, mapping, or the crack index (4). This latter term is a measure of the cracking severity and is calculated by adding the number of full transverse and 1/2 the number of half transverse cracks occurring in a 500-foot stretch of two-lane pavement.

Fracture of the pavement surface arises when the tensile stresses exceed the strength of the surfacing material. Two broad categories of cracks have been observed, produced by two distinctly differing mechanisms. In the first, the crack appears in the bituminous surface; in the second, the crack extends into the subgrade occasionally penetrating the shoulder and adjoining soil foundation. Although the second mechanism involves subgrade soil shrinkage, it is not included in the analysis of induced tensile stresses under low temperature. Induced stresses in the bituminous surface due to forces arising from underlying movements are a key concern in pavement overlays.

Study of the first mechanism in instrumented field test projects has indicated that the initiation of cracking is at the time of prolonged low temperatures and minimum seasonal values (5, 6). Haas and Topper (7) have discussed these and other postulated mechanisms in great detail.

Low-temperature shrinkage cracking occurs because decreasing temperatures have a shortening effect on the asphalt concrete surface. At very low temperatures, the normally viscous fluid asphalt cement binder tends to become a solid and ceases to permit sufficient plastic flow to accommodate this shrinkage. As a result, the pavement ribbon develops tensile forces within and transverse cracks across the surface. The flow properties of the asphalt concrete and the shrinkage are temperature and rate dependent. A method has been described to assess the critical temperature at which shrinkage rate exceeds the viscous flow (4).

More rigorous analytical stress analyses have been performed and compared with observations of two test projects in western Canada (8). Stress predictive methods, coupled with the use of appropriate temperature and materials characterization data, provide a means of assessing the low-temperature fracture susceptibility of asphalt concrete paving mixtures.

DESIGN APPROACHES TO COUNTER TRANSVERSE CRACKING

Review of literature and current experience indicates that design approaches may be categorized as

1. Setting limits on asphalt specifications,
2. Setting limited stiffness values for asphalt or asphaltic concrete mix,
3. Predicting fracture temperatures, and
4. Predicting cracking frequency.

Setting Limits on Asphalt Specifications

From the results of early field studies (1) involving field inventories and data gathering, the low-temperature fracture susceptibility of an asphaltic concrete mixture appears to depend on the asphalt used. After recognizing this dependency, various highway departments modified their asphalt specification in an attempt to reduce the occurrence of transverse cracking. These modifications included setting higher penetration values and/or implementing minimum viscosity requirements. Typical of such modifications were those introduced during the mid-1960s by the provinces of Saskatchewan and Alberta.

Prior to 1963, the Saskatchewan Department of Highways asphalt specification required 150-200 penetration grade asphalt. Subsequent modifications led to the minimum and maximum penetration and viscosity requirements at 77 and 140 F respectively. Until 1967, asphalt cements used in highway construction in the province of Alberta were graded according to penetration at 77 F resulting in asphalt cements of a given penetration grade exhibiting large differences in viscosity at 140 F. In 1967, the Alberta Department of Highways introduced its current asphalt specification, which incorporates a minimum viscosity requirement at 140 F.

The grade most commonly used calls for a minimum penetration of 250 at 77 F and a minimum viscosity of 275 poises at 140 F. This specification has resulted in a more uniform product from several distinct asphalt sources, the elimination of tender mixes and an apparent reduction in low-temperature transverse cracking under normal winter conditions (9).

Methods of selecting asphalt cements on the basis of penetration and temperature susceptibility characteristics, in the light of Ontario experience (10), are shown in Figures 1 and 2. The charts advocate the use of softer asphalt grades and/or less highly temperature-susceptible asphalt cements for locations where low winter temperatures (-40 F or -40 C) may be expected (3).

In Ontario, softer asphalt grades (300-400) have been in use in the northern areas for more than 5 years. Most of the pavements constructed with this soft binder are on granular base. All the mixes have been designed by a method similar to the Marshall mix design method (11). The mixes have conformed to the stability criteria and, in the 5 years, have shown no signs of pushing, rutting, or instability.

It should be stated that none of these roads with the softer asphalts is very heavily travelled. A softer, slow curing asphalt SC-5 grade binder has been widely used in Manitoba over a long time period on heavily travelled routes without rutting or pushing distress (5).

The use of softer asphalt grades in pavements in northern Ontario has delayed the onset of transverse cracking by 3 to 5 years and has reduced the frequency of cracking to a tolerable level.

Whereas highway agencies have reported a reduction in transverse cracking through the use of softer asphalts, the influence of these asphalts on traffic load associated distress, such as rutting and fatigue, has not been fully evaluated. Densification under traffic together with noticeable embedment of some chip seal coats is indication of possible distress, particularly where extremes of low temperatures may require the selection of very soft asphalts.

A great deal of controversy has developed around the grading of asphalt cements by penetration at 77 F or viscosity at 140 F (or 275 F); however, it has become apparent that the grade and the temperature susceptibility must be considered together for any particular climatic condition. The temperature susceptibility as described by the pen-vis (PVN) instead of the penetration index (PI) appears to be a very practical method (12).

Setting Limited Stiffness Values for Asphalt or Asphaltic Concrete Mix

McLeod concluded that transverse cracking will occur if the asphaltic concrete paving mixture, at the minimum service temperature encountered, falls within the range of 1×10^6 to 2×10^6 psi (2). He suggested that these values were applicable for a dense, well graded mix and asphalt stiffness values derived from his 1969 modification of Van der Poel's stiffness nomograph at a loading time of 20,000 seconds. Fromm and Phang determined the stiffness of asphalts of various mixtures known to exhibit different low temperature crack frequencies and from this suggested a limiting stiffness of 20,000 psi at a loading time of 10,000 sec (4) (Figures 1 and 2).

In 1972 the British Columbia Department of Highways introduced an asphalt specification that had a limiting asphalt stiffness as its basis (13). By using similar nomographic procedures and a loading time of 7,200 seconds, stiffness values were computed for a number of pavements across Canada. Cracking did not occur when the stiffness value of the bitumen at a service temperature of -40 C was less than 29,000 psi. This critical stiffness was converted to penetration values at two temperatures in order to produce a workable specification.

Although these methods are really intended as estimates in arriving at a satisfactory selection for the asphalt cement, it should be recalled that all are based on indirect methods of determining asphalt or mix stiffness and, accordingly, appreciable differences may result between methods (3). In addition, other factors such as particular environmental conditions, subgrade, and material factors may have an overriding effect. This may be the reason why the Saskatchewan experience has shown that cracking did occur at -40 F within the criteria given in Figures 1 and 2 within the first 12 months of service (14).

Predicting Fracture Temperatures

Basically this is a variation of the stiffness approach in that stiffness values are used to calculate the thermally induced stresses within an asphaltic concrete paving mixture subjected to a particular temperature history. The predicted stresses are compared with the fracture strength-temperature relationship of the mix in order to estimate the fracture temperature (1, 3). In a similar manner, the critical temperature (4) may be determined directly by creep testing under various temperature conditions.

In both approaches the intent is to determine whether conditions are likely to occur where tensile stresses exceed the strength of the asphalt surface and transverse cracks form.

Predicting Cracking Frequency

Hajeck (15) has developed a mathematical model for predicting cracking frequency as a function of several variables. The model is based on field data from the provinces of Ontario and Manitoba and has undergone extensive evaluation in terms of statistical significance, rational behavior, and relation to the observed data (3). This model enables prediction within a reasonable degree of confidence and has the capacity to provide an immediate engineering guide for controlling low-temperature cracking in Canada. Developed from data on pavements that had not been overlaid, the usefulness of this model in pavement rehabilitation is therefore limited.

PROPOSED DESIGN GUIDELINE

Christison has proposed a simplified approach that enables the influence of variations in asphalt mix stiffness, asphalt stiffness, and volume concentration of aggregate on induced thermal stresses to be readily deduced (8, 16). Combined with information on tensile strengths for the particular case, the likelihood of fracture can be estimated.

The development of the guideline involved the following computations.

1. By using the psuedo-elastic beam analysis and a loading time of 7,200 seconds, thermal stresses within each of the asphaltic concrete paving mixtures were computed when subjected to five temperature histories. These temperature histories included the three recorded at the Manitoba test project during the 1967-68 winter, the measured asphaltic concrete pavement temperatures at the Alberta test project during the winter of 1966-67, and the predicted third winter temperatures at this test site. For all stress predictions the coefficient of thermal expansion of each paving mixture was assumed constant and equal to a value of 1.5×10^{-5} per degree Fahrenheit.
2. At temperatures of 0 F and less and with 5-degree increments, maximum thermally induced stresses within each of the asphaltic concrete pavements were transcribed from the results of the stress analyses. At corresponding temperatures and a loading time of 7,200 seconds, stiffness values of the paving mixtures were computed. The maximum predicted thermally induced stress values were then plotted as a function of mix stiffness values. Individual stress and stiffness values of the low viscosity 150-200 asphalt pavement at the Manitoba test project are shown in Fig. 3, with the bands in this figure representing the limits of the thermally induced stress-stiffness relationships of all asphaltic concrete mixtures considered.
3. The relationship among asphaltic concrete stiffness, asphalt stiffness, and volume concentration of aggregate propsed by Heukelom and Klomp (16) is shown in Fig. 4. Superimposed on this figure are the limits of the maximum predicted thermally induced stress-mix stiffness relationships shown in Fig. 3.

Fig. 4 provides a means of readily estimating thermally induced stresses within asphaltic concrete pavements subjected to low-temperature environments. Such estimations assume that the unknown thermal stresses can be approximated by the use of pseudo-elastic beam analysis and that the coefficient of thermal expansion of paving mixtures equals 1.5×10^{-5} per degree Fahrenheit and is temperature-independent. Because predicted stresses are directly proportional to a temperature-independent expansion coefficient, their position may be shifted along the upper horizontal axis of Fig. 4 when a different coefficient is used.

At a given temperature, the stress estimate requires a knowledge of asphalt stiffness at a loading time of 7,200 seconds together with the appropriate volume concentration of aggregate

Figure 1. Selection of asphalt cement grade for various design temperatures (12).

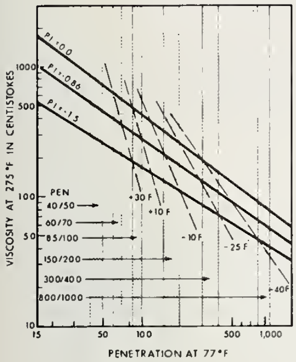


Figure 2. Selection of asphalt cement grade for various winter design temperatures, 1 % basis.

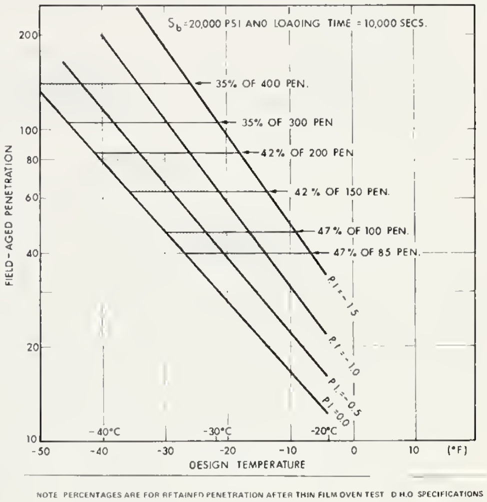


Figure 3. Stiffness modulus of asphaltic concrete versus maximum predicted thermally induced stress.

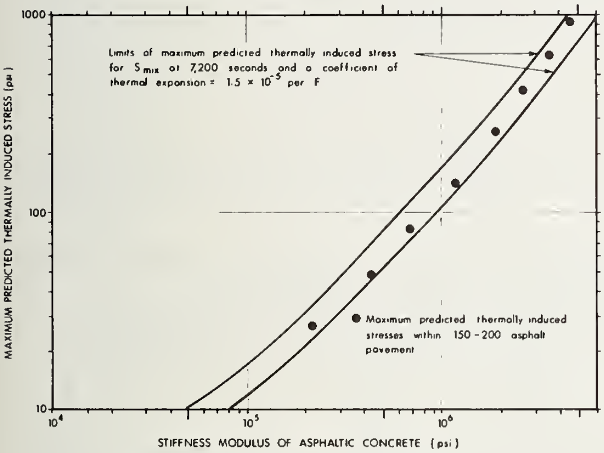
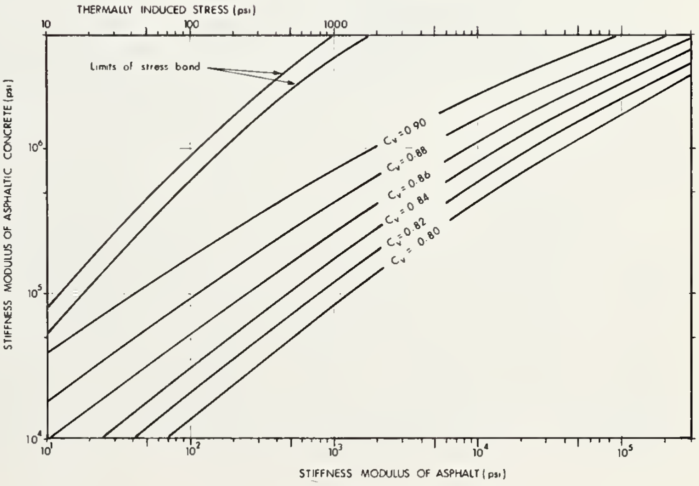


Figure 4. Relationships between asphalt and asphaltic concrete stiffness moduli (16).



value (C_V) for the paving mixture or a knowledge of the asphaltic concrete stiffness at the same loading time. In the former case, the importance of defining the C_V value of the paving mixture is shown in Fig. 4. From this figure, relatively small variations in C_V can result in large differences between predicted thermally induced stresses in a paving mixture whose asphalt stiffness has been defined.

Low-temperature tensile strength failures for asphaltic concrete mixtures composed of western Canadian crude sources and local aggregates range from approximately 200 to 500 psi. Comparisons have been made with the previously described limiting stiffness criteria, and reasonable agreement has been achieved.

REFLECTION CRACKING OF OVERLAYS

Prior to the introduction of softer asphalt grades, pavements in Ontario with transverse low-temperature shrinkage cracks were overlaid with asphalt mixes containing either 85-100 or 150-200 penetration grade asphalt cements. Experience has shown that reflection cracking of the overlay occurs during the first winter after completion of the overlay. Trials of single and double surface treatments over transverse reflection cracks have also failed during the first winter. Filling the reflection crack with a hot poured rubberized joint filler has proved unsuccessful. The jointing material separates from one side of the crack during the winter, and moisture and dirt entering this gap prevent readhesion the following summer. Additionally, this treatment is quite expensive, and no lesser joint filling operation has shown any signs of success.

The reason for the failure of asphalt overlays and joint filling operations may be gleaned from the result of measurements of the movement of cracked pavement edges during the winter. Fig. 5 shows typical plots of the horizontal movement of transverse cracks at two locations. At one location the crack measured 0.23 and 0.22 inch in the winters of 1961-62 and 1962-63 and had residual openings of 0.05 and 0.02 inch after the crack closed during the summer. At the other more northern location, the crack measured 0.56 and 0.64 inch in two consecutive winters, and there were residual openings of 0.21 and 0.17 inch in the 2 years, i.e., a total residual of 0.38 inch in 2 years.

Measurements during the winters of 1961-62, 1962-63, and 1963-64 of transverse cracks at five locations revealed that cracks measured as much as 0.81 inch and that the mean crack opening varied from 0.20 to 0.35 inch over the three winters. The residual openings were negative in two cases but were as much as 0.21 and had mean values of 0.07, 0.04, and 0.05 inch over the three winters (17).

For cracks 1/2 to 1 inch wide, the crack opening amounts to 50 to 100 percent increases. Any material bonded to the crack must be expected to undergo this amount of extension.

Data presented by Rix (18) on pavement crack-width measurements in Alberta during 1964 to 1967 show a similar trend. The transverse crack width opens from 0.1 to 0.3 inch during the winter and there is a residual opening of about 0.1 inch over three winters (Fig. 6).

REFLECTION CRACKING MECHANISM

When an asphalt pavement with transverse shrinkage cracks is overlaid, the old pavement, at a relatively lower level in the pavement structure, is subjected to temperatures appropriate to that depth. Analyses of data from field projects (5, 6) have shown that, at the minimum daily air temperature, the temperature of the surface at increasing depths is progressively warmer. The rate of temperature change is also attenuated at increasing depths. The cracking mechanism described previously can still be applicable; however, a more severe condition may be operative.

As previously indicated, measurements of movement at transverse cracks show that the cracks widen during the winter and tend to close during the following summer. The opening of the crack could be considered somewhat like the end of slab effect in a portland cement concrete pavement. It is speculated that the amount of the opening of the crack is related to the expansion or shrinkage coefficient of the asphalt concrete, the lowest temperature encountered during the winter, the critical temperature (4) of the asphalt concrete, thickness of asphalt layer, and the condition of base restraint (Fig. 7a.

The mid-points of two adjacent slab portions could be considered immovable because of the base restraint. Isolating the section between these two points with a new continuous surface overlaid on the cracked old surface can be shown in Fig.7b. With a drop in temperature, the tendency of the old pavement to shorten (crack opening) will set up restraint stresses on the underside of the new surface, resulting in high tensile stresses directly above the crack as shown in Fig.7c. If these stresses are sufficiently high, fracture will result.

The tensile stress developed because of underlying slab movement is related to the degree of restraint. With a bonded overlay and therefore a high degree of restraint, the entire movement is forced to be accommodated over a length of approximately 1 inch. With end of slab movements in the order of 1/2 inch, this would require a strain capability of some 50 percent. This is impossible in a normal asphalt concrete.

Figure 5. Typical horizontal distance data summary charts (71).

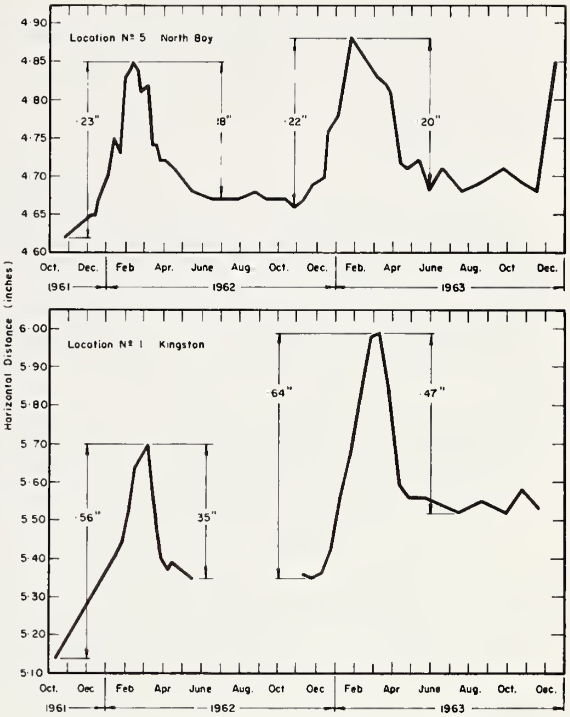


Figure 6. Seasonal patterns in width of transverse cracks.

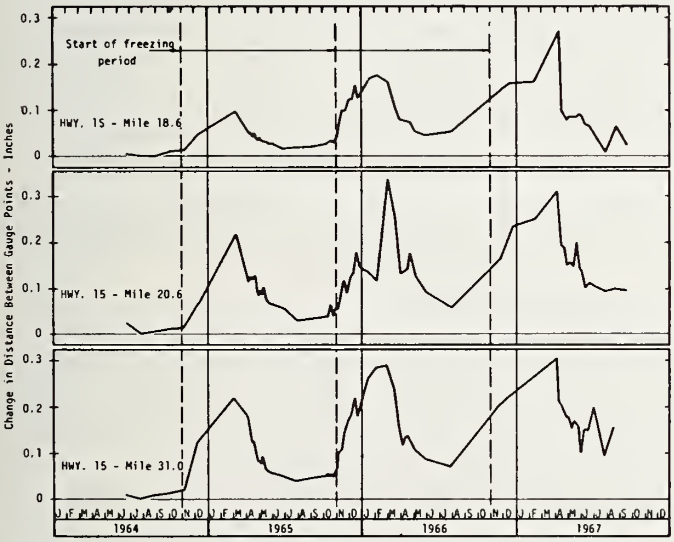
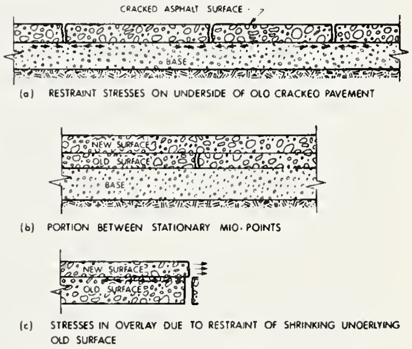


Figure 7. Bituminous pavement overlay.



A possible solution to this problem would be to reduce the degree of restraint in order to effectively lengthen the section over which the slab movement is accommodated.

The suggested mechanism of shrinkage cracking due to subgrade drying, described by Uzan et al (19), may have some application to this problem as well. Their theoretical analyses may be useful in establishing the mechanical properties of the surface and any intervening layer to prevent reflection cracking.

OVERLAYS

Reflection cracking in overlays is a problem of great concern to all highway personnel. Throughout the years many treatments have been tried, with varying degrees of success. Much of the effort to minimize or preclude reflection cracking has been with asphalt concrete surfacings over soil-cement bases and with bituminous overlays over old concrete pavements. Descriptions of various treatments are discussed by Norling (20). Descriptions of wire mesh reinforcement for bituminous overlays of old concrete pavement are given by Brownridge (21), by Graham (22), and in a manual issued by the Wire Reinforcement Institute (23). Open-graded base course mixes were used over old concrete pavement in Ontario in 1968 and in Arkansas (24).

Reflection cracking in bituminous overlays over old asphalt pavements has also received attention from McDonald (25) and Gallaway (26).

The treatments that have so far received attention are summarized below

1. Overlay mixes (a) with increased asphalt binder content, (b) with softer asphalt binders, (c) with thick asphalt films (open-graded), (d) with mastic asphalts, (e) with rubber and polymer asphalt additives, and (f) with asbestos fibre fillers;
2. Reinforced overlays;
3. Intervening layers such as (a) granular materials and (b) open-graded asphalt mixes; and
4. Stress relieving interface (SRI) such as (a) rubber aggregate mixes and (b) asphalt/polymer.

Overlay Mixes

Use of asphalt concrete mixes with high asphalt contents have been unsuccessful because of problems of flushing or bleeding. In conjunction with asbestos fiber fillers, however, these mixes apparently are sometimes successful in reducing reflection cracking (27). Trials with asbestos in asphalt mixes have been conducted in Ontario (4), but there was little difference in transverse cracking behavior observed between mixes, indicating that other factors were obscuring the effects of the asbestos. Nevertheless, laboratory tests indicate promising qualities (28).

Mixes with softer asphalt binders have been tried in Ontario over a stretch of Highway 17 at Hayden (about 10 miles north of Sault Ste. Marie). This highway had transverse low-temperature cracks at intervals of 5 to 10 feet. The asphalt concrete overlay, placed in 1969, contained a soft 300-400 penetration grade asphalt cement. Because the cracks were so closely spaced and were about 1/8 to 1/4 inch wide, the softer asphalt binder permitted sufficient flow to accommodate the shrinkage movements in the old pavement even at the very low temperatures (-30 or -35 F expected at the location. Even this soft grade of asphalt binder could not prevent reflection cracking; hairline cracks appeared over practically every transverse crack in the old pavement during the early part of the first winter.

The very high asphalt content of mastic-like mixes such as Gussasphalt (29) hold some promise of ability to withstand reflection cracks; however, this material has received only limited trials in North America (30). Extensions due to low-temperature shrinkage can be very high, and it appears unlikely that this material, by itself, can meet this requirement.

Attempts to minimize reflection cracking of overlays on old concrete pavements by the use of thick asphalt films on open-graded base mixes have been fairly successful in Ontario. In 1968, on a section of Highway 401 about 10 miles west of Toronto, an open-graded base course mix was laid over an undowelled concrete pavement whose 20-foot slabs were faulted between 1/2 to 1 inch. The mix consisted of coarse aggregate only, maximum size 1 inch, with 3 percent 85-100 penetration grade asphalt cement. This 1-1/2 inch thick base course was covered with two lifts (3-1/2 inches) of regular design asphalt concrete.

Although some hairline cracks appeared immediately under roller compaction, the cracks have not widened and the overlay appears to be performing satisfactorily. This same type of mix was used in 1959 on a composite pavement test section further west on Highway 401 (31). After 13 years the cracks in the surfacing had widened to about 2 inches. The AADT on Highway 401 is now 29,000.

Hensley in 1973 indicated that open-graded mixes with 2 to 3 inch maximum size stone have been used successfully in Tennessee and Arkansas to prevent reflective cracking over both old concrete and old asphalt pavements (24).

McDonald described mixtures consisting of 20 to 30 percent of ground tire rubber and 80 to 65 percent of asphalt binder mixed at temperatures of 300-500 F (150-260 C), applied at 0.3 to 1 gallon per square yard followed immediately with an application of cover aggregate, as preventing the reflection of fatigue or alligator cracking; the treatment will not entirely prevent reflection of large shrinkage cracks (25). He also described the application of ready made band-aid types of patches of this material to alligatored areas.

Reinforced Overlays

Brownridge, reporting on installations of wire mesh reinforcing of bituminous overlays over old concrete pavements in Ontario, concluded that, in the more southerly parts of the Province, "continuous welded wire mesh has substantially reduced the incidence of reflection cracking in the first five years of service" but, in the low winter temperatures of northern Ontario, the continuous welded wire mesh is not effective in controlling transverse cracking. "The horizontal movement due to temperature changes are more likely to cause reflection cracking than vertical movements due to heavy loads" (21).

Graham (22) reported that wire mesh reinforcement of bituminous overlays in New York retarded reflection cracking on old concrete pavement with joints at less than 80 feet for up to 4 years. The mesh reinforcement was not effective in other situations.

Intervening Layers

The use of intervening granular layers originated in New Mexico. The upside-down design, as it is known, is used extensively in New Mexico, Arizona, and British Columbia over cement-treated bases (20). The treatment consists of 4 to 6 inches of untreated granular base material placed above the shrinkage crack-prone cement-treated base and below the asphalt concrete surfacing. A 1/2 to 5/8 inch plant mix seal coat is placed either at the time of construction or a few years later.

Whereas this treatment does not prevent reflection cracking entirely, the cracking under New Mexico conditions does not appear for 3 to 5 years and is narrower and further apart than normal. The treatment is reported to be very successful in one part of British Columbia where temperatures range from 29 to 72 F (-2 to 22 C) but not so successful in projects where larger temperature ranges prevail.

A similar principle of using an intervening layer, although asphalt stabilized with an MC cutback asphalt, is used in Alberta where cement stabilized bases are used (32). Experience has shown that not all the cracks that occur during the initial 2-week curing period are reflected through the 2-1/2 inch asphalt bound base into the asphaltic concrete surfacing. Although documented evidence is not available, experience during the mid 1950's indicated that overlays of asphaltic concrete placed on thicker asphalt bound bases, in the order of 4 to 6 inches, tended to have few reflected transverse cracks.

Stress Relieving Interlayers

The stress relieving interface (SRI) concept described by Gallaway (26) in 1971 was made practical in Texas by the use of a slurry of rubber tire aggregate particles of 1/8 inch maximum size, sand to provide good gradation, and asphalt emulsion. A field application of this material was described by James (33).

Laboratory data indicate that a 1/4 inch layer of SRI could be expected to accommodate 0.20 inch of base movement with a minimum of reflection cracking. Gallaway suggested that overlays of 2 to 3 inches of asphalt concrete might be adequate over old concrete pavement if SRI is used.

A similar concept is suggested in the literature by a major asphalt supplier (34). This strain-relief interlayer is recommended for use in sophisticated flooring applications where a long life is required without surface cracking. It consists of about 15 percent of a thermoplastic rubber mixed with asphalt cement and filler.

THE TROUT CREEK EXPERIMENT

For an idea of how different treatments work to retard or prevent reflection cracking, an experiment consisting of eight test sections was constructed on Highway 11 at Trout Creek in 1971 (35, 36). This section of highway, located some 18 miles south of North Bay, Ontario, was due to be resurfaced because of the very rough ride (especially during the winter period) caused by the presence of transverse cracks. The cracks were spaced 20 to 50 feet apart, were generally 1/2 inch wide, and depressed 1 to 2 inches over a 4 foot (1.2 mm) spread. Transient lipping or tenting was reported during the winter.

The asphalt concrete overlay mix was a standard HL-4 mix (well graded) with a 150-200 penetration grade asphalt cement. Details of gradation and mix qualities are given in Table 1. The paving operations were carried out between August 21 and October 8, 1971.

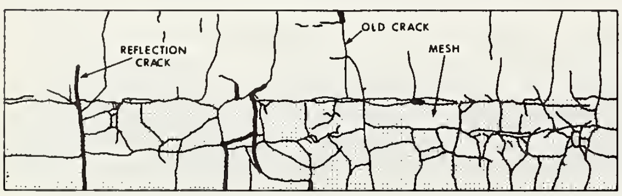
Section 1

A 1-inch layer of granular 'D' screenings was spread over the old pavement and shoulders to a width 2 feet wider than the old pavement. Because of distortions in the old pavement, it was not practical to keep to the 1 inch specified. The rubber tires of the asphalt paver disturbed the

Table 1. Mix data for Trout Creek experiment.

Sieve Size	OPEN-GRADED BASE % Ret.	STANDARD HL-4 (Binder & Surface) % Ret.
5/8 in.	1	1
1/2 in.	11	5
3/8 in.	60	27
3	82	40
4	91	45
200	98.5	97.9
Asphalt Content	3.4 percent	5.7 percent
Stability, lbs.		2110
Flow,		10.4
Voids, %		2.5

Figure 8. Pavement crack map of nonmetallic mesh reinforcement in overlay.



layer excessively, and the initial section had to be removed. Construction went more smoothly (and at lower speed) with the second attempt but the lower asphalt binder course was increased to 2 inches instead of 1-1/2 inches. A 1-1/4 inch upper binder course and a 1-1/4 inch surface course were subsequently placed. The length of this section was 500 feet. No reflection cracking of any kind was observed during the winters of 1971-72 and 1972-73.

Section 2

This section has 3 inches of a granular 'A' base material placed on the old pavement and over the shoulders. To ensure the required thickness, the first run of the box spreader was made along the centerline of the pavement; the rest of the cross section was then brought up to the necessary cross falls. No construction problems were encountered. The overlay was made up of two lifts of binder (1-1/2 and 1-1/4 inches) and 1-1/4 inches of surface course.

No cracking was observed during the winter of 1971-72, but slight transverse cracking (6, of which 3 are part cracks) was observed in 1973. This section is 0.67 mile long.

Section 3

This treatment is similar to that of section 2, except that the granular 'A' layer is 6 inches thick instead of 3 inches and laid in two lifts. The granular interlayer was extended over the full width of the pavement and shoulders. The asphalt concrete surfacing was 4 inches laid in three lifts.

Two transverse cracks were observed during the winter of 1971-72; this increased to 5 after the 1972-73 winter, and crack widths were about 1/4 inch. This section is 0.67 mile in length.

Section 4

In this section the interlayer is an open-graded mix with 5/8 inch maximum size coarse aggregate only, containing 3.4 percent asphalt cement. The mix was laid by paver, 1-1/2 inches thick, compacted with steel and pneumatic-tired rollers, and allowed to cool to about 120 F before traffic was permitted to use it. No ravelling under traffic was observed, but the 1-1/4 inch upper binder course of standard mix was placed the following day; one 1,000-foot section was left uncovered for 2 days before being covered, with no detrimental effects due to traffic. The surfacing course of 1-1/4 inch standard mix was placed finally. This section of 1.22 miles was 36 feet wide because it started and ended a truck climbing lane.

Grooves, 1/2 inch deep by 1/2 inch wide, were cut at intervals of 25, 50, and 75 feet and also random intervals. The grooves were filled with hot poured rubberized jointing compound.

After one winter's exposure, 89 reflection cracks appeared. Ten percent were hairline width; the rest were 1/4 inch. Only 8 of these cracks were within 10 feet of a sawn groove; it was concluded that grooves were of no benefit in controlling reflection cracking under these conditions. After the second winter, 1972-73, the number of transverse cracks had increased to 147, and 10 percent of these were now wider than 1/4 inch.

Section 5

The treatment here is really for control. The old asphalt pavement was scarified and removed. The broken up pavement was put through a jaw and cone crushing and screening unit (one inch maximum size). The granular material from the base, which was removed togher with the old asphalt, was replaced with fresh granular base material. The crushed old asphalt was used to bring up the shoulders to the finished pavement level. Because of the need to maintain traffic, only one lane of the old pavement at a time could be broken up. The new pavement surface over the old granular base consisted of the three lifts of standard mix totalling 4 inches. The length of this section is 1.08 miles.

After the first winter, one transverse low-temperature shrinkage crack was observed; this was close to one of the sawed joints (similar to those in section 4). After the second winter, the number of transverse cracks increased to four; two cracks were now near to sawn grooves.

Section 6

The treatment was to pulverize the old pavement to a 1 inch maximum size and reuse this crushed material as the new base. The old pavement was first broken up by a ripper and graded into a windrow. A Bros preparator was used initially to pulverize the 1 foot chunks of old pavement, but several passes were required. The preparator needed extensive maintenance and eventually quit.

To complete the job, the remainder of the pulverizing was carried out by the crusher located at the gravel pit about 1 mile away. No difficulty in crushing was encountered. The screened material was relaid on the old granular base and compacted and graded to form the new base. This material worked well, even in fine grading prior to paving, and formed a good base for the asphalt layers. These were the 3 lifts of standard mix, totaling 4 inches. The length of this section was 1.0 mile.

After the first winter no transverse cracks were observed, but there was one longitudinal crack caused by settlement in a swampy area. This settlement became more pronounced the second winter, but, again, no transverse cracks appeared.

Section 7

The treatment here is similar to that in section 6, except that the pulverized old asphalt pavement was plant mixed with a liquid asphalt binder before it was relaid as a treated base for the new asphalt pavement. Again the old asphalt pavement was ripped up, passed through a jaw and cone crusher, and screened. The crushed material was heated slightly only in the dryer of a regular asphalt plant; greater heat tended to cause plugging up of the hot elevator and hot bins. It was then mixed in the pugmill with 4 percent of an MC-250 and laid by paver. This binder content was later reduced to 3 percent.

The newly laid and compacted mat would not set up and exhibited severe distortion and tracking of aggregate under traffic. It was windrowed aside and airing with a pulvimixer which hastened curing time. The cured material was then windrowed back and spread with the grader. While there was still slight distortion under traffic, this disappeared with the first binder course of standard mix. Two other lifts of standard mix was laid to provide the total of 8 inches of new asphalt concrete surfacing. This section is 0.9 mile long.

No cracking of any kind has been observed after two winters.

Section 8

This section was a control section. The three lifts of new asphalt were laid directly over the old cracked pavement.

Dips in the old pavement were corrected with a sand mix before laying the lower binder course. As with the other section, the total overlay thickness was 4 inches. The length of the section is 1.1 miles.

After the first winter there were 94 transverse reflection cracks; 10 percent were hairline, the remainder were approximately 1/4 inch wide. The cracks were not really near any of the sawed grooves. After the second winter in 1972-73, the number of transverse cracks increased to 150 and 10 percent of these are now wider than 1/4 inch.

Conclusions

The granular interlayers used in treatments 1, 2, and 3 were effective in reducing the number of transverse reflection cracks in the new surfacing. The new pavement on old granular base was also effective in reducing the occurrence of low-temperature transverse cracks. The open-graded mix used as an interlayer between old and new asphalt surfacing was not effective in reducing transverse reflection cracking; there remained 120 transverse cracks per mile as compared with the 136 transverse cracks per mile in the standard overlay treatment of section 8.

The two treatments that successfully resisted the onset of transverse cracking for two winters were those in which the old asphalt pavement was pulverized and reused as a base either with or without added liquid asphalt binder. This method is likely to be suitable mainly for thinner asphalt pavements but might prove to be economical even for thicker asphalt pavements.

OTHER EXPERIMENTS

An experiment using nonmetallic mesh reinforcement for the overlay was carried out over a badly cracked pavement in St. Boniface, Manitoba, in 1969 (37). The reinforcement is a 1/2 inch square mesh made up of strands of polystyrene plastic material and is quite flexible but not very extensible. The mesh was laid down on a cured tack coat and held in position by fresh hot mix. It was laid 10 feet wide in two 6-foot overlapping strips over a length of 80 feet. The overlay consisted of 2 lifts of asphalt concrete totaling 4 inches. The asphalt cement was high-viscosity 150-200 penetration grade.

Transverse reflection cracking appeared in January and March of the following winter (Fig. 8). The nonmetallic mesh has initially reduced the amount of transverse cracking.

Burlap strips fastened to the old asphalt pavement prior to the placement of the asphalt overlay was tried in 1968 in Manitoba (38). All cracks reflected through.

Eighteen-inch wide strips of thin rubber laid over transverse cracks on the Trans Canada Highway at St. Anne, Manitoba, in 1968 showed promise of preventing reflection cracking; however, similar tests in 1972 on PTH 75, Manitoba, did not confirm this promise as cracks reflected through within six months.

SUMMARY

Low-temperature shrinkage cracks can be prevented or retarded in new pavements by the use of softer and less temperature susceptible asphalt cement binders. While there has been little rutting experienced with these softer asphalt binders, the addition of asbestos fibers will apparently stiffen the mix at high temperatures and reduce the temperature susceptibility of the mix even at low temperatures.

Softer asphalts will not prevent or retard cracking caused by shrinkage of the subgrade during low temperature conditions; these occur at specific locations and are limited in extent.

In cracked asphalt pavements, as in jointed concrete pavement, the horizontal movement due to low-temperature shrinkage is concentrated at the edges of the slabs, joints, or cracks. Any overlying layer that is bonded to the cracked pavement must endure a concentration of elongation directly above a transverse crack of about 50 percent. No bituminous overlay material has successfully resisted the stresses imposed by this horizontal movement although wire mesh reinforcement does tend to retard the formation of reflection cracks.

A stress relieving interlayer made with rubber aggregate shows promise of minimizing reflection of shrinkage cracks under Texas climatic conditions. The method deserves trial under more extreme temperature ranges.

Possibly a solution to the reflection of low-temperature shrinkage cracking lies in a combination of one or more of the different treatments described previously. For example, a stress relief interlayer placed over a transverse crack like a band-aid in conjunction with reinforcement of the asphalt overlay in the vicinity of the crack may be one solution to this problem; at the same time it may not be a practical solution. The influence of the thickness of the overlay on its ability to resist reflection cracking must also be considered.

At this time, in areas such as all Canadian provinces where very low temperatures are experienced, old pavement with numerous transverse cracks are probably best broken up and reused as a base for new asphalt concrete surfaces. This treatment has many advantages including conservation of aggregates, less strength loss in spring, stiffer base at all times of the year, less softening at cracks in the surfacing, and less susceptibility to volume change due to frost action. Because of the magnitude of the problem of reflection of thermal cracking, a specific research effort is needed to arrive at solutions that are practical and economical. Because the effects of thermal cracking are felt most keenly in Canada and the northern United States, it is in these environments that possible solutions must be evaluated. Work on stress relief layers and on pulverization of existing layers appears to hold definite promise.

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DRAINAGE AND PAVEMENT REHABILITATION

George W. Ring

Good surface and subsurface drainage systems are needed in the original construction of a road to ensure that a pavement will last its design life. It has been said that poor subsurface drainage, combined with frost action, can result in the temporary loss of two-thirds of the strength of flexible pavements and one-third of the strength of rigid pavements (1). Heavy loads applied during this period of strength loss can greatly shorten the life of the pavement.

Unless properly designed, original drainage systems can become inoperative by the time rehabilitation is required. In fact, gradual reduction in effectiveness of poorly designed drainage systems can be the primary cause for premature pavement distress. Even when the original drainage system remains in good working order, construction operations during pavement rehabilitation can change conditions, rendering the drainage system either inadequate or inoperative. Blocked drainage paths, water trapped between old and new layers, clogging of open-graded materials, and lengthened drainage paths in widened pavements are examples of problems that can be created during rehabilitation resulting in water-logged pavement structures. Finally, improvement of a roadway can result in an increase in traffic loadings making the original drainage incapable of preventing early damage to the rehabilitated pavement.

The main purposes of pavement rehabilitation are to

1. Provide a new, smooth riding surface
2. Cover up defects in order to prevent further deterioration such as rutting, longitudinal waviness, cracks
3. Reseal porous pavements
4. Provide a wider surface
5. Present a skid-resistant surface
6. Strengthen the pavement structure.

The most desired result is a smooth, high-speed surface, with an extended lifetime because of the added strength of the overlay. Determining the required pavement rehabilitation or overlay thickness to ensure this extended life is difficult because of both lateral and longitudinal variations in pavement strength. Pavement edges are often weaker than the interior because of less lateral support and wetter subgrade conditions at the edge after a rainy period. Similarly, water collecting in the pavement structure at vertical sag curves can cause a problem in the longitudinal direction. Other sources of lateral and longitudinal variations include changes in soil type and construction joints and differences in weather, materials, and contractor operations during construction. In dealing with these nonuniform conditions, efforts should be made to balance solutions to structural problems with increase in overlay thicknesses (Figure 1). This is not intended to imply that adding greater thicknesses of overlay can always solve the structural and drainage problems. Indeed, the practice of simply piling on thick layers of resurfacing without considering basic problems, such as drainage, should be expected to have only a temporary, cosmetic effect in those sections having either very weak subgrade support or free water trapped within the pavement structure. If we are to find a balanced solution to the rehabilitation problem, we need more information than just the response of a pavement system to loads. We need to understand the causes of response variation from one section of pavement to another. Only then can we make an engineering decision that will result in a balanced solution. For example, if one section of pavement shows consistently low load capacity, we should make an effort to determine whether we can raise the load capacity of this section by some method other than adding thicker overlays. It may be more economical in the long run to use a combined solution of a thinner overlay along with remedying a basic drainage problem.

Drainage problems to be considered in pavement rehabilitation sometimes occur over long stretches of a project and at other times are restricted to local areas.

Examples of general problems often extending over long sections of pavements are

1. Shallow side ditches
2. Blockage of subsurface drainage due to widening
3. Permeable shoulders and medians
4. Pumping rigid pavements
5. Impermeable aggregate drainage layers
6. Reduction of drainage capacity of curbed pavements due to overlays
7. Water in open-graded bases (trench section)
8. Drainage of open-graded plant mix seals

Shallow Side Ditches

Shallow side ditches are usually associated with older roads having narrow rights-of-way, but occasionally are the result of pavement widening in cut sections. The problems caused by shallow ditches are water-softened shoulders and free water within the pavement structure (Figure 2). Resurfacing tends to raise the pavement grade but does not affect the level of water in the subgrade and critical parts of the pavement structure as long as the ditches remain shallow. Because flexible pavements derive much of their support from the subgrade, it is especially important to their good performance that side ditches be made deeper where needed and cleaned of debris and vegetation to permit rapid runoff of water. Sometimes the material removed when deepening ditches is suitable for building up unpaved shoulders to overlay height. When material removed from the ditch is of poor structural quality, it is usually wasted.

Blockage of Subsurface Drainage due to Widening

When rehabilitation involves widening, existing daylighted layers and pipe drain outlets in shoulders and ditches may be inadvertently blocked. Also, when permeability of the base course in pavement widening is much different from the permeability of the original pavement, water traps can be created at the old and new base course interfaces (Figures 3 and 4). The solution is to consider permeability in the design of base course widening and ensure that the permeability will always be increasing in the direction of water flow.

Widening may also divert other drainage so that new sources of water are directed into the pavement structure.

Permeable Shoulders and Medians

Unpaved shoulders and medians should always be considered as a potential source of water for the pavement structure (Figure 5). Paving the shoulders has often proved beneficial for increasing the life of pavements. Some of the benefits are increased lateral support of the pavement edge and reduced infiltration of surface water. When a pavement is overlaid, it is usually necessary to raise the shoulder. Where the original shoulders are unpaved, consideration should be given to bringing them up to grade by paving.

Pumping Rigid Pavements

Rigid pavements that pump before being overlaid are likely to resume pumping afterward unless (a) subsealing and/or mudjacking is effective,(b) the overlay is thick enough to prevent reflective cracking or considerably reduce deflections, and (c) internal drainage is improved.

Subsealing with blown asphalts reduces the water reservoir space under the pavement and has been partially successful in controlling pumping. Past experience indicates that injection points should be closely spaced for the treatment to be effective (2). Mudjacking is more often used to correct the grade of faulted and depressed pavements. Because mudjacking does not necessarily correct a basic drainage problem, pumping is likely to gradually resume after a short period of time (3).

Faulted rigid pavements are sometimes overlaid with asphaltic concrete to improve their smoothness with subsequent development of reflective cracking of the joints. A major problem with reflective cracking, aside from unsightliness, is that even small cracks can admit very large quantities of surface water into the pavement structure, thereby reducing the support capacity of both the subgrade and the pavement structure. To keep the water out requires that the overlays not develop reflective cracking. However, very thick overlays are required to completely prevent reflective cracking. The Asphalt Institute recommends 4.5 inches (4). The California Division of Highways recommends 3.6 inches for slab lengths of less than 20 feet (5). Thicker overlays are needed to prevent reflective cracking over longer slab lengths. Even these recommended thicknesses do not completely prevent reflective cracking for the design life of the overlay. When thick overlays are necessary to prevent reflective cracking and are not needed for structural load support, thinner overlays with improved drainage should be considered as an alternate solution.

Impermeable Aggregate Drainage Layers

Slow-draining aggregate base courses are found on nearly all existing roads. Even those few made of fast-draining materials often have no outlets. Where base courses do have outlets, it usually takes a long time for water to reach them. Cedergren found that it took from 20 to 2,000 times longer for water to drain from conventional aggregate bases than from hydraulically efficient open-graded drainage blankets (6). Many state specifications for granular drainage layers will permit materials having permeabilities as low as 0.01 foot per day. However, to achieve successful drainage, the permeability of drainable bases should be from 10,000 to 100,000 times higher than the conventional slow-draining base courses. These much higher permeabilities are easily achieved through small changes in gradations in the design and construction of base courses in new pavements. In existing pavements, solving the problem of slow draining bases is not as simple. However, the scope of the problem can be reduced by intercepting water at joints and edges, before it has a chance to get under the pavement.

The great attention being given to slow-draining bases in existing pavements indicates that this problem is far from being solved.

Reduction of Drainage Capacity of Curbed Pavements due to Overlays

In many northern states and especially in suburban areas, pavement edges are often lined with curbs so that surface water is drained into drop inlets. When the pavement grade is raised by overlays, water is more likely to spill over the curbs onto the shoulder and eventually find its way into the pavement structure. Suggested solutions are to raise the curbs or to pave the shoulder to make it less pervious.

Water in Open-Graded Bases

Water in open-graded bases is usually the result of pavements built in trench sections with either no drainage outlets, or outlets having much less outflow capacity than the inflow rate. Inflow rates are sometimes underestimated since sources of water include surface infiltration as well as ground water seepage. Infiltration through cracks and joints, porous pavements, and unpaved medians and shoulders are probably the largest sources of water in many pavements. Construction during the dry season can also result in underestimation of inflow rates. Support capacity of pavements with saturated base courses is reduced by at least 25 percent (7) and more where there is a tendency for the water to flow upward, such as occurs with groundwater seepage and by solar heating of nearly saturated base courses (8). Greater losses in strength are also associated with thawing of ice in frozen bases and subgrades.

Solutions to the problem of water in open-graded bases consist of plugging inflow sources as much as possible, strengthening elements of the pavement by adding stabilizing binders, and constructing outlet drains. Because joint and crack sealing is seldom continuously effective, and outlet drains sometimes become clogged, good sealing practices and close attention to drainage are needed to obtain the best guarantee of good pavement performance.

Because most infiltration of water into rigid pavements and thermally cracked flexible pavements is likely to occur at transverse joints, cracks, and pavement edges, some states have installed drains from the edge joints through the shoulder (Figure 6). This practice is often effective in reducing pumping. However, most base courses under existing pavements are relatively slow to drain, and water infiltrating at transverse cracks and along the edges will take a long time to reach joint drains. For this reason, longitudinal drains at the pavement edge, tied to transverse drains through the shoulder, are more effective (Figure 7) than joint drains alone. One state is installing edge drains with laterals on 20-foot intervals tied into an existing longitudinal pipe underdrain in the shoulder (Figure 8). This approach is being used where water is encountered under the pavement in patched areas and where edge pumping is serious. The cost is about $1.75 per linear foot. Where the original shoulder is in poor condition, reconstruction of the shoulder with a drainable layer may be possible. Louisiana is experimenting with this type of reconstruction (Figure 9) by using an asphalt-treated open-graded drainage layer similar to that advocated by O'Brien and Cedergren (6).

Drainage of Open-Graded Plant Mix Seals

Use of open-graded plant mix seals is becoming a popular method for obtaining a high level of skid resistance and for reducing splash and spray from trucks. These seals do their job by providing channels for surface water to drain to the side of the pavement away from direct contact with tires. They are quite effective when this drainage is not impeded; however, use of these seals where the side drainage is blocked, such as by a curb or lip, may reduce their effectiveness. Excess asphalt in these mixtures appears to help seal the original surface.

Generalized stability problems extending over long areas can be detected by an analysis of conventional deflection measurements, by overall shortened pavement life, or by observations. Localized problems are generally more obvious because they are usually characterized by short stretches of rough and broken pavements. Examples of localized problems are

1. Pockets of frost-susceptible soils
2. Pockets of unstable subgrade

3. Broken and clogged pipes and outlets
4. Frozen water in depressions on old pavement
5. Raising of inlets for overlays

LOCALIZED PROBLEMS

Pockets of Frost-Susceptible Soils

Because pavement roughness due to frost heave usually subsides after the frost leaves the ground, normal roughness and deflection evaluations during summer seldom identify localized frost problem areas. Where they are identified, it should be noted that only free water can be drained from frost-susceptible soils. Because of their high capillarity, these soils will still contain a considerable amount of water even when well drained. Lowering of the free water table by installing drains might help to reduce the supply of water that can be pulled into the cold zone. However, it is often more effective to replace small pockets of silty soils than to drain them (Figure 10).

Pockets of Unstable Subgrade

These areas can be identified by localized pavement distress. Common causes are peat pockets, localized springs and groundwater seepage, inoperative subsurface drainage systems, sinkholes in limestone areas, and pockets of open-graded subgrade surrounded by less permeable soil. For localized springs and groundwater seepage, installation of drains is effective. Peat pockets and nonuniform soil problems are best solved by replacement, although mixing of nonuniform soil deposits is sometimes successful.

Broken and Clogged Pipes and Pipe Outlets

Broken and clogged pipes can act as underground water storage tanks and even as dams. Careful preliminary survey and subsequent reporting of all damaged pipe outlets should be made a standard part of the pavement rehabilitation process. In some instances, repair of pipe outlets may be the only remedy needed. However, inoperative pipes caused by clogged pipe perforations or joints under the pavement are not easily detected or repaired. Although back flushing may be effective, dry drain outlets should be suspect, and if poor drainage is obvious from the condition of the road, cleaning or replacement of underdrains may be necessary. Careful marking of pipe outlets with posts or outlet structures will help to reduce future damage by maintenance equipment.

Frozen Water in Pavement Depressions

When pavement overlays consist of an asphaltic concrete with substantial void spaces, water tends to collect in the mixtures at depressions and sags in the old pavement. In cold climates, the freezing water can separate the layers and even cause rupture of the resurfacing mixture.

Raising of Drop Inlets for Overlays

Because overlays raise the pavement elevation with respect to drop inlet grates, the inlets become more hydraulically efficient. However, if the difference in elevation becomes too great, there can be a safety problem and the inlets must be raised. Where the original grates are unable to accommodate normal water flow, replacement with parallel bars in the direction of flow can result in increased inlet capacity. These grates pose a problem for bicycle trafiic. The FHWA is testing a new type of grate having transverse bars set at an angle, which may be a good compromise between hydraulic efficiency and safety for bicyclists (9).

DESIGN AND CONSTRUCTION RESPONSIBILITY

In most overlay projects, state maintenance forces perform surface and subsurface corrective work including drainage remedies, spot repairs, and replacement of pavement sections. When repairs are completed, the overlay is usually placed by a paving contractor with little input from state design engineers. In instances where rehabilitation involves relocation and widening, recommendations of state soil and hydraulic engineers can be incorporated in construction contracts.

Figure 1. Balanced design for rehabilitation.

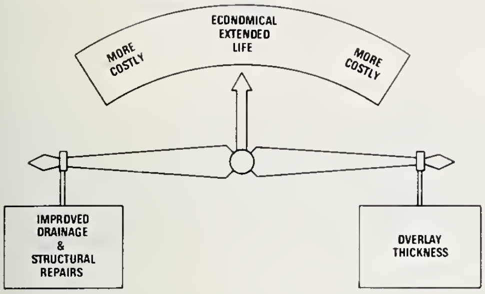


Figure 2. Shallow ditches cause soft shoulders.

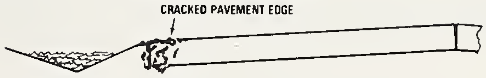


Figure 3. Trapped water in widening.

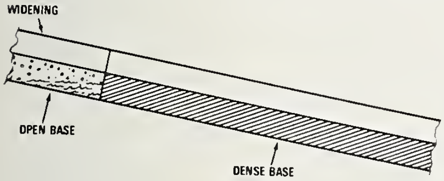


Figure 4. Trapped water in original pavement.

Figure 5. Porous shoulders admit water to pavement structure.

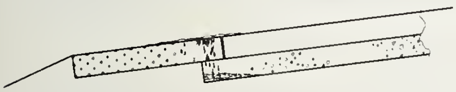


Figure 6. Drains at joints and cracks.

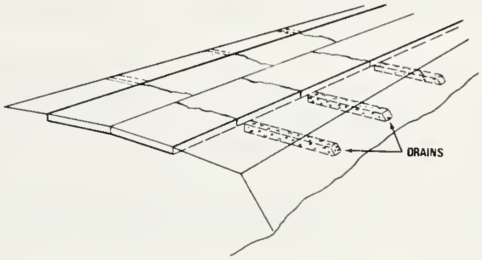


Figure 7. Edge and joint drains.

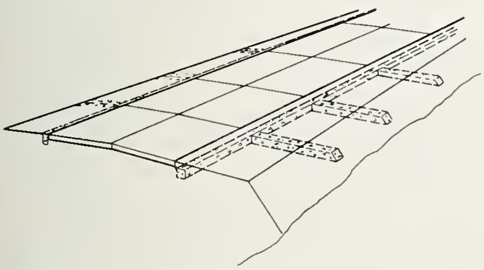


Figure 8. Utilize existing drains.

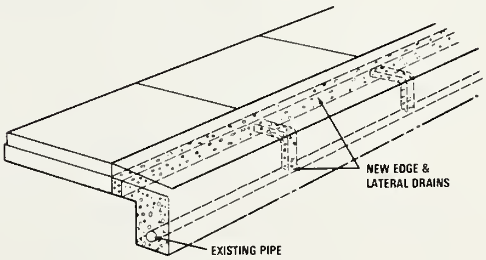


Figure 9. Drainage through shoulder.

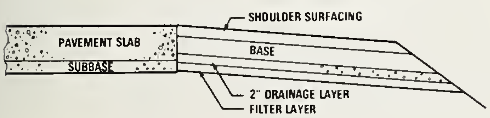
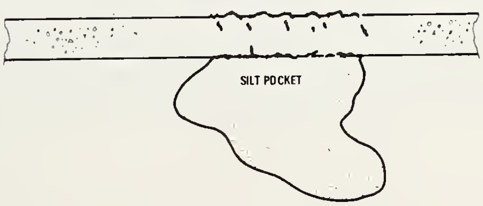


Figure 10. Pavement damage by frost action.



SUMMARY

The Asphalt Institute (4) has posed four general questions that should be answered by surface and subsurface drainage investigations for pavement rehabilitation:

1. Is the original design adequate for drainage of the existing road?
2. What changes in design are necessary to ensure that drainage inadequacies which may be a contributing factor to structural distress, are corrected?
3. If the original drainage system was adequate, have environmental or structural changes taken place since it was built that require reconstruction of the system?
4. Does present or projected land use in areas adjacent to the road indicate that surface drainage flow patterns have changed or are likely to change, thus rendering existing drainage facilities inadequate?

Answers to these questions provide the opportunity for a balanced design in pavement rehabilitation. It appears that there is a need for participation of state soils, pavement and hydraulic engineers in more rehabilitation projects, especially since a substantial part of the more than $4 billion budgeted each year for maintenance and traffic operations is spent on pavement rehabilitation.

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URBAN AREA PROBLEMS ASSOCIATED WITH PAVEMENT REHABILITATION

L. G. Byrd

As the freeways and major arterials in urban areas mature, many agencies face extensive pavement rehabilitation programs in order to maintain service on these critical facilities. Typically, urban systems are operating at or near peak capacity for many hours each day, and the prospect of performing pavement rehabilitation is a formidable one.

In addition to high traffic volumes, urban systems often are subjected to axle loadings in excess of legal limits. This is brought on, in part, by the difficulties experienced by regulatory agencies in attempting to control axle loadings for short-haul urban trips where weigh stations are not provided and where the opportunities for portable scales are limited. In northern communities, where snow and ice are a factor, the concentration of traffic volumes and use of studded tires have caused grooving of wheelpaths. With heavy axle loads and high volumes of traffic, the service life, in many instances, of new pavements in urban areas is less than 10 years.

Many urban routes are located within limited rights-of-way where options to detour traffic over median areas, paved shoulders, or side turf areas are limited or nonexistent. Because of the density of development, overpasses and pedestrian bridges impose clearance restrictions at short intervals. Subsurface storm drainage with complex inlet and collection systems that are an integral part of pavement or gutter sections in many urban areas must be considered during pavement rehabilitation.

Urban freeways and arterial systems have other characteristics of importance to pavement rehabilitation. Aerial space over the highway may be occupied by buildings, and areas beneath elevated highways may serve as parking areas, parks, playgrounds, buildings, or other surface facilities. Median strips may be occupied by rail transit systems. The trend toward multimodal use of rights-of-way and using aerial rights, and surface areas beneath roadways will be further encouraged as our limited urban land areas are reluctantly assigned to transportation corridors in the future.

Problems associated with pavement rehabilitation in urban areas can be divided into three categories: planning, design, and construction problems.

PLANNING PROBLEMS

Because of the complex interrelationships of the various departments within highway agencies, many have adopted the task force concept for the successful development of urban pavement rehabilitation projects. The task force usually includes representation from planning and programming, design, traffic engineering, construction, maintenance, and public information. The task force defines the objectives of the project and reviews the design concepts, construction plans, method for handling traffic, and proposed public information program. In a broad sense, the task force monitors the project from its inception through contract letting and, in some instances, through construction.

In addition, when a pavement rehabilitation project is planned, there must be careful coordination and cooperation with a large number of external agencies. Most urban freeway systems serve in a transportation network under the responsibility of many jurisdictions. Because the activities performed on one segment of the network have repercussions on all other segments and all other transportation facilities within the urban area, coordination is essential.

Regular meetings and reviews, are needed with other local governments, police and fire departments, ambulance service organizations, public transit companies, utility companies, automobile clubs, and trucking organizations. Planning also is needed to coordinate the rehabilitation project with those organizations sponsoring public events such as baseball games, major theater productions, parades, or other traffic-generating activities within the urban area.

Traffic handling considerations represent one of the major planning problems of urban roadways. With detailed traffic information in hand, alternative procedures and staging of the rehabilitation project can be explored and the impact of the alternative proposals assessed. The State of California has developed procedures for determining the optimum time to close a freeway lane and for estimating delay to motorists when congestion occurs at a lane closure.

Fig. 1 shows a cumulative curve for vehicles using a freeway over a period of several hours. When compared with a cumulative capacity curve for a two-lane closure beginning at 9 a.m., cumulative vehicle demand would exceed the cumulative capacity by 980 cars with a maximum delay of 19 minutes per vehicle by 10 a.m..A project starting at 10 a.m. would cause no delay.

Fig. 2 provides another approach to delay analysis during a pavement rehabilitation project. When a known or observed volume per lane and length of back-up at the construction site, the delay time per vehicle can be determined by reading horizontally from the volume to the curve then vertically to the appropriate length of back-up scale.

Figure 1. California cumulative vehicle and capacity curves used to compute delays at lane closure sites.

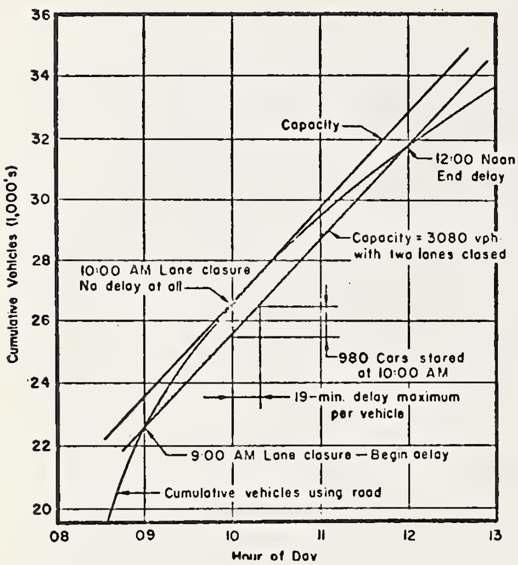


Figure 2. California volume-delay curves for computing delay at lane closure site.

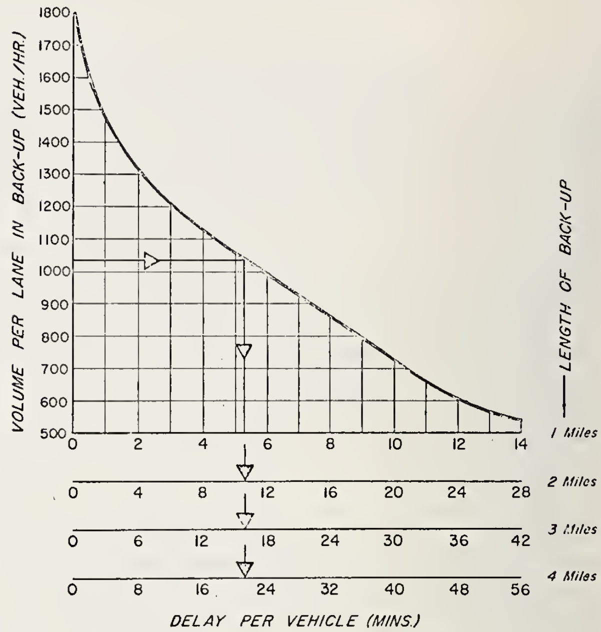
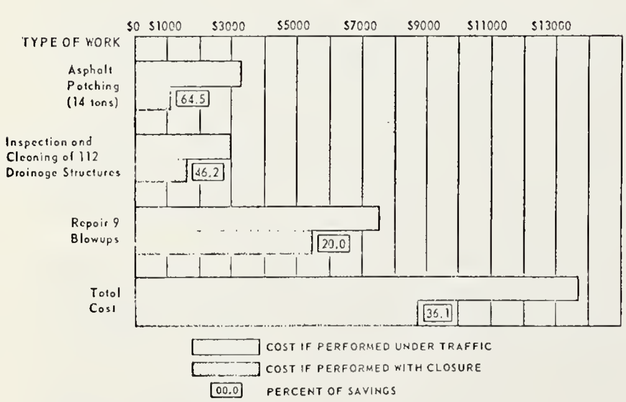


Figure 3. Cost and traffic comparison for northbound John C. Lodge Freeway (1968 dollars).



An alternative to lane closures or restrictions that has been used with success in Detroit is the complete closure of segments of the freeway system during night hours with detour routes established on parallel streets. Fig 3 shows an estimated construction cost savings of 36 percent by closing the Lodge Freeway rather than performing the work under traffic. Fig. 4 shows the closure and detour plan employed on the freeway. From the evaluation of the freeway closures in Detroit it was concluded that

1. Closures are best accommodated from mid-June through August and mid-week from 10 p.m. to 5 a.m. Weekend closures are best from 2 a.m. until 10 a.m.
2. Ramps serving as entrance and exit points around a freeway closure should be able to handle at least two lanes of traffic. Two-lane capacity can be provided in some instances by using paved shoulders and temporary marking materials to mark lane lines.
3. Entrances to the freeway system within a mile of the closure area should be closed so that traffic does not enter the freeway system only to be diverted back onto the local street system within 1 mile.

On-site detours within the right-of-way are another alternative to lane closures or lane restrictions. Where right-of-way width permits, temporary new pavements and structures adjacent to existing pavements and structures can be constructed. The New Jersey Turnpike uses this technique and attempts to provide geometrics on the temporary detour pavements comparable to those on the original roadway. More common is the use of paved shoulders as supplemental pavement lanes or the reversal of lanes in the opposing traffic roadway. Where shoulder pavements have been used as temporary lanes, some agencies have attempted to confine the use of the shoulder lane to passenger traffic by advance signing requesting that passenger vehicles only use shoulder lane. Under heavy urban mixed traffic volumes, this sorting-out of vehicles is sometimes difficult to achieve. The practice suggests strong warrants for the original construction of freeway shoulder pavements capable of carrying normal traffic loads. Quite often the original construction cost of a pavement and shoulder section built to a uniform standard is no greater than the cost of construction of separate and different pavement and shoulder sections.

A number of agencies have successfully used lane reversals in conjunction with median crossovers, although some freeway operations groups have experienced problems with this procedure. When lane reversals are used it is necessary to provide a significant physical delineation of opposing traffic flows. One barrier used effectively for this purpose on the Illinois Tollroad (Fig. 5) is constructed of precast sections of concrete of the New Jersey or GM safety shape. The New York State Department of Transportation and some other agencies are using two stacked 12 x 12 timber sections fastened together at end points and pinned to the pavement at third points.

Public Information

One important factor contributing to the success of major freeway rehabilitation projects has been the use of an effective information program. A successful public information program was carried on by the Illinois Department of Transportation on a project involving a $16 million resurfacing of major expressway in metropolitan Chicago 15 miles long. The program included 12 carefully planned steps covering

1. Meetings with other highway agencies and interested public and private transportation organizations,
2. Pre-bid meetings with contractors,
3. Formal briefings with the communications center dispatchers, the expressway emergency patrol supervisory staff, expressway surveillance personnel, and maintenance yard supervisors and individual briefings with radio traffic reporters from the major municipal radio stations,
4. Two formal press conferences held by the Secretary of Transportation,
5. Requests to private companies responsible for large changeable message advertising signs adjacent to the expressway to publicize the starting date of the project.
6. Cross-street bridge-mounted signs indicating that expressway repairs begin on given date.
7. Traffic engineers accompanying helicopter traffic reporters to ensure the availability of traffic control explanations,
8. Weekly status reports issued throughout the project, and
9. Signs thanking the motorists for their cooperation erected at the conclusion of the project.

A public information program of this nature will contribute to the orderly control of traffic in the working area. There are other supplemental considerations that experience has suggested to several agencies. One is the advantage of beginning a project on a mid-week day rather than on Monday morning. Detroit found that the commuter forgets about traffic control information

Figure 4. John C. Lodge Freeway closure and detour plan for pavement rehabilitation project.

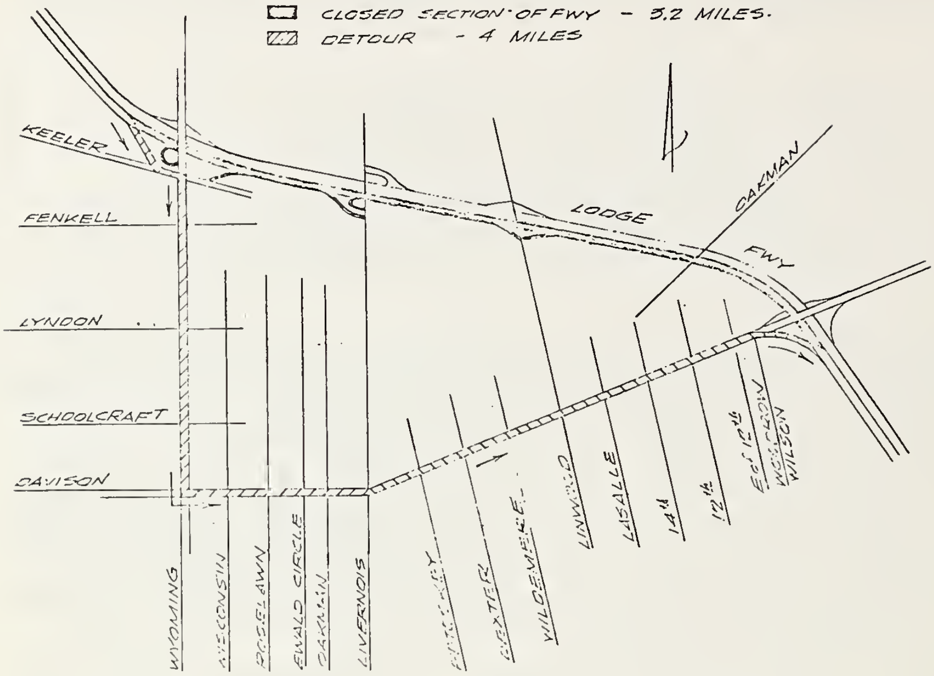
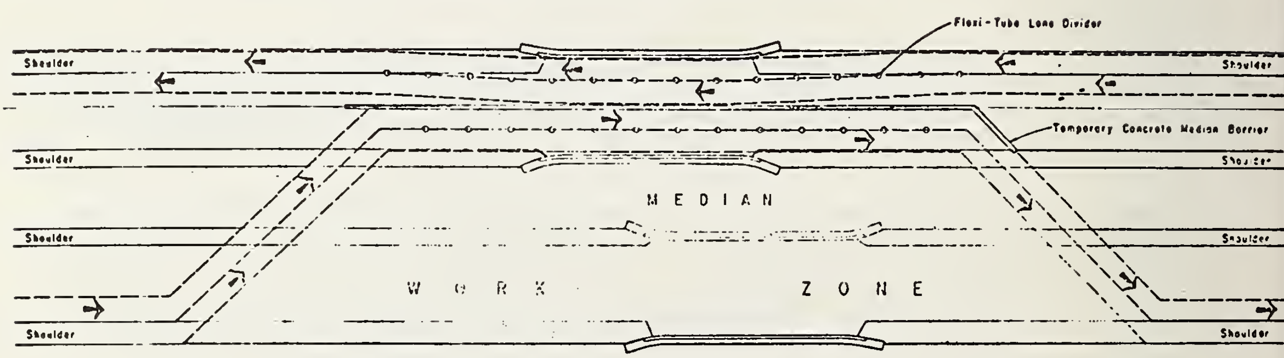


Figure 5. Illinois tollway lane-reversal plan using precast concrete.



DETAIL OF FLAG CREEK BRIDGE RUN-AROUND

disseminated during the prior week. The motorist is more likely to retain radio messages and sign messages along the route if he hears and sees them the day before the project begins. Further, mid-week days usually produce a somewhat lighter traffic volume than Monday mornings, and thus the reaction to barricades and traffic constrictions is less severe on a mid-week morning.

DESIGN PROBLEMS

When the design for a pavement rehabilitation project in an urban setting is developed, two sets of constraints must be accommodated. One is imposed by the time limitations during which the work must be performed. If the work must be done during off-peak daylight hours with a maximum of 5 or 6 hours of project site occupancy available at any one time, then the materials selected and the design employed must be functional within these time limits. Where detours can be provided for long-term occupancy of the project site, options to use slow curing materials or longer term construction procedures are available. Restriction of the work to nighttime or off-season periods may add further constraints.

The second set of constraints is caused by the multiplicity of structures and appurtenances found on urban arterial or freeway sections. Overhead structures impose clearance restrictions. If several inches of new pavement overlay are added to the existing pavement profile, inadequate clearances may result. Two options have been used to avoid this clearance problem. One is lifting the bridge deck by jacking and placing higher pedestals on piers and abutments to increase clearances. As an alternative, pavement sections under overpasses have been partially or fully removed so that a rebuilt section can be constructed to the original pavement surface elevation without loss of overhead clearance. Both of these options are costly and time-consuming.

Overlay pavements, if carried across bridge decks, may represent an unacceptable increase in dead loads on the structures. Where this is the case, pavement rehabilitation projects may require a design that terminates the overlay pavement at either end of the bridge deck. The taper of the new overlay pavement at the bridge approach to meet the deck profile can be achieved by removing existing pavement material or by tapering the thickness of the overlay pavement. Most agencies are in agreement that pavement tapers must not result in a featheredge ending if the taper section is to retain its stability and integrity under service.

Curb and gutter sections with inlets and other drainage facilities can create further design problems for overlay pavements. If overlays are carried onto the gutter section, they may obstruct drainage inlets and reduce the hydraulic capacity of the gutter section. In urban areas where most drainage runoff is controlled and collected for subsurface transportation, violation of gutter capacities and other drainage features may create major problems.

Guardrail, delineator, and sign elevations may be adversely affected by pavement overlays. Changing the elevation of these appurtenances may be necessary to restore the relationship between the pavement surface elevation and the elevation of these features.

One other design problem involves realistic assessment of design axle loads. With high volumes of commercial vehicles on urban freeways and arterials and with frequent overloads undetected on short-haul urban trips, the importance of selecting appropriate loading criteria for the design of the rebuilt pavement must be recognized.

CONSTRUCTION PROBLEMS

The special traffic handling requirements encountered during the reconstruction of urban pavements represents one of the major construction problems. Plans for on-site and off-site detours, for lane closures, signing, signals, and lighting, are developed in detail as part of the design phase of rehabilitation projects. During the construction phase, implementation of these design concepts usually requires on-site assistance from experienced traffic engineering, maintenance, and police personnel.

Many urban highway agencies use special teams of police officers with experience in handling traffic at construction sites. These teams, assigned to construction sites during rehabilitation projects, observe and evaluate the design concepts to make sure that traffic controls are functioning properly and handle traffic as transitions are made from one lane closure to another or from standard operations to lane reversals.

On the New Jersey Turnpike, the state highway patrol teams assigned to construction operations have developed a technique whereby a platoon of patrol cars can merge with the traffic flow upstream of a construction site and gradually form a moving barrier to slow down traffic. This maneuver is used several miles in advance of a construction site to create a time gap in the traffic flow sufficient for moving heavy equipment across the roadway, erecting overhead beams, or other action that would otherwise require traffic to be blockaded. By creating a gap in the traffic flow, the patrol squad provides time for these crossroad activities to be completed, often without stopping traffic. This same technique has been used by the police patrol to create a traffic gap when traffic is being diverted from one lane closure to another.

Police patrol vehicles or other specially marked vehicles have been employed as pilot vehicles to lead traffic along a new detour route being opened.

Construction problems frequently are encountered in planning a project to accommodate traffic, particularly where work time is restricted to offpeak hours or nighttime. Contractor schedules must be carefully developed and followed in order to provide finished, cured surfaces available for traffic when peak-hour flows must be accommodated. The moving of heavy equipment, forms, materials, and other supplies to and from the job site during short time intervals with precise schedules imposes a condition on the contractor that frequently leads to difficulties.

Time restrictions compound construction difficulties where the limited right-of-way in urban areas does not accommodate on-site storage areas for materials and equipment.

Environmental concerns pose an important problem for construction of pavement rehabilitation projects in urban areas. Noise and air pollution restraints have limited the use of piledrivers, airhammers, and other major construction equipment units. Burning restrictions based on air pollution concerns have prevented the use of heater planers and other burning techniques for pavement cutting and repairs in some urban areas, e.g., Los Angeles. The problem of waste disposal is becoming acute in some areas especially the removal of deteriorated concrete, steel, and other waste materials from a rehabilitation project.

Typical of the environmental considerations that must be included in current construction programs are the growing concerns over dust and sulfur dioxide emissions in asphalt plants and air pollution resulting from the dissipation of volatiles in asphalt cutbacks. A number of agencies, including the Wisconsin DOT have implemented restrictions on sandblasting as a result of air and water pollution from the dispersal of waste sand and metal oxides removed by the sand. Michigan has a statewide regulation requiring that tarpaulins or hard covers be fixed on all open-body maintenance trucks to prevent hauled materials from falling or being blown onto the roadway or into the air. Dust from shoulder blading and pavement sweeping represents a violation of air pollution regulations in many areas today; these regulations require use of water or dust palliatives as a part of such programs. The noise problem is compounded by nighttime maintenance operations when noise levels are closely regulated in urban areas.

SUMMARY

The urban area problems associated with pavement rehabilitation, can be placed in three categories: those problems caused by time and traffic relationships, by space and work relationships, and by the environment and pollution relationships. None of the problems is unique to urban areas, but all are present to some degree and all are accentuated by the urban environment. When it is realized that urban areas are where most pavement rehabilitation work is urgently needed, the importance of dealing with pavement rehabilitation under the constraints of the urban problem is easily recognized.

PAVEMENT REHABILITATION
HIGHWAY MAINTENANCE PROBLEMS

Travis Smith

As one charged with the maintenance of a highway system, I would like to see the development and construction of maintenance-free highways, although I realize this is not possible for a variety of reasons. It would also be highly desirable to develop an information system that would predict when maintenance was needed on a section of roadway, what work was needed, how it should be performed, and how much it would cost. We should also have assurance that money would be available to do the work. However, we live in the real world that does not have these information systems or the history of projects, soils, loads, and other variables necessary to make such a system work. Therefore, we evaluate, plan, budget, maintain, and repair roads by using rather inexact methods in the face of monetary, political, and emotional constraints.

In this discussion of maintenance strategies and pavement rehabilitation, I shall draw from my experiences as maintenance engineer in California. With some variations, I expect that my comments will be valid for most of the states.

The statutes of the State of California define maintenance as

1. The preservation and keeping of rights-of-way and each type of roadway, structure, safety convenience, or device, planting, illumination equipment, and other facility, in the safe and usable condition to which it has been improved or constructed, but does not include reconstruction or other improvement.
2. Operation of special safety conveniences and devices and illuminating equipment.
3. The special or emergency maintenance or repair necessitated by accidents or by storms or other weather conditions, slides, settlements, or other unusual or unexpected damage to a roadway, structure, or facility.

The statutes further provide that "the degree and type of maintenance for each highway, or portion thereof, shall be determined in the discretion of the authorities charged with the maintenance thereof, taking into consideration traffic requirements and moneys available therefor."

To give some idea of the magnitude and complexities of the problems, the California state highway system consists of 34,632 lane miles of asphalt concrete pavement and 14,323 lane miles of portland cement concrete pavement. This includes many miles of heavily traveled freeways and ranges down to some rather primitive two-lane rural roads. The newer roads are generally well constructed, engineered sections, whereas many of the older roads were built 40 to 50 years ago and their structural sections are unknown. Many of them were not engineered but were built up from gravel placed on the bare ground. (Some of these early roads were constructed by the state, while others were constructed by the counties and later taken into the state highway system.) Climatic conditions vary from the extreme heat of the desert valleys to the heavy snows of the mountain ranges and the high rainfall areas on the coast. Traffic varies from very light in rural areas to a high of more than 200,000 vehicles per day on a segment of freeway in Los Angeles. All of these factors and others contribute to the variety and complexity of highway problems and our efforts to cope with them.

As time and utilization factors work on our roads, pavements oxidize, become brittle, and crack; water enters the base and subbase, resulting in distortion with further cracking of the surface; faulting develops in our PCC pavements, to the dismay of truck drivers particularly; pavements become slick, ravel, pothole, settle, and heave, ad infinitum.

To combat these defects and fulfill our obligations to maintain the roads, we have a number of options or programs available to us.

First of these options are the activities of our maintenance crews in general maintenance. These include the usual activities of crack sealing, surfacing and patching, seal coats, base and surface repair, mudjacking, and variations of these tasks.

Table 1 contains the 1972-73 fiscal year production units and dollar costs for these activities.

Table 1

Maintenance Activity	*Flexible Roadbed Production Units	Dollars	**Rigid Roadbed Production Units	Dollars
Crack Sealing	7,600 (100#CTN)	370,000	12,000 (100#CTN)	370,000
Surfacing & Patching	580,000 Tons	9,800,000	20,000 Tons	620,000
Seal Coats	50,000 Tons	2,000,000	1,000 Tons	150,000
Base & Surface Repair	140,000 Tons	2,680,000	8,000 Tons	210,000
Mudjacking	--	--	69,000 Sq. Yds. Raised)	575,000
Totals	--	14,850,000	--	1,925,000

*Asphalt Concrete or Portland Cement Concrete Pavement with more than 2 inches of asphalt cover.

**Portland Cement Concrete.

Another option is major maintenance. This consists of maintenance tasks that exceed $5,000 for a specific job. These are financed through regular maintenance funds but are submitted as a specific program and approved prior to start of the work (they are included in Table 1 under surfacing and patching).

In addition to maintenance activities, we have the thin blanket program, financed by construction funds but managed by the maintenance department. This program is budgeted at $12,000,000 a year and consists of 1-inch overlays of asphalt surfacing, placed by contract, on carefully selected segments of the road system. The objective of the program is to extend the useful life of the road by adding new, relatively impervious, smooth layers of surfacing to the roadway. The base must be sound and the existing surfacing reasonably intact for this treatment to be effective. Approximately 1,000,000 tons of asphalt surfacing are placed annually under this program.

Other alternatives include the use of structural blankets and major reconstruction of the roadway section. These are more costly methods of restoration and are budgeted as major projects when funds are available.

With limited resources and information on which to base decisions, how do we make our choice of the options available for maintaining the highway system? It is not through sophisticated computer systems but with great reliance on the judgment of our people in the field.

Each year a budget is adopted, dividing the state's available highway funds among planning, rights-of-way, new construction, and maintenance. The funds available for maintenance are further divided among the 11 transportation districts on the basis of their highway inventory and demonstrated needs. Each foreman then develops a work plan for the highways in his territory. The superintendents make necessary adjustments and combine the work programs of their foremen. This forms the basis of our statewide maintenance program. There are, of course, deviations from this work program as dictated by emergencies and weather conditions.

In addition to the maintenance work program, each district maintenance engineer makes recommendations for some portions of roadway to receive thin blankets and for others to have major repairs. These recommendations are reviewed by management and then fitted into the construction work load with consideration for funds and other limiting factors previously discussed.

EVALUATING ROAD CONDITIONS

To assist in the choice of rehabilitation options, we use several methods of evaluating road conditions. Our transportation laboratory uses either the traveling Deflectometer or Dynaflect to test AC pavement deflection (the Deflectometer is generally preferred because of more consistent results).

This equipment is used to determine the amount of pavement deflection and the size and shape of the deflection area. The values are used in evaluating pavement condition, predicting service life, and recommending corrective treatment.

The Dynaflect, a self-contained mobile unit, is towed behind a vehicle, usually a pickup. In the test position the Dynaflect exerts a 1,000 pound peak to peak oscillatory load on the pavement surface by means of two steel test wheels. Based on a correlation established between this equipment and the traveling Deflectometer, a recommendation for reconstruction can be made. Dynaflect measurements are usually taken every 0.01 mile, or at 53-foot intervals, in the outer wheel track of each test section.

Another aid is our pavement evaluation program. In 1969 the maintenance department formed a committee to develop a pavement surface condition rating system in order to better manage the maintenance program.

After studying what other states had done and determining what was needed to meet California's needs, the committee developed our system.

Beginning in 1969, two statewide pavement evaluation surveys have been made, and the pavement condition reports are being used to budget and plan maintenance resources.

In our pavement evaluation system we assign point values to varying types of pavement defects on a length of asphaltic concrete road. We then total the points, producing a sum of defects that we can compare with another length of road. Utilizing a PCA road meter, we determine a ride score and combine it with the sum of defects to give us a single number, called the pavement condition rating. We evaluate all 34,000+ lane miles of asphaltic concrete pavement in lengths ranging from 1/2 to 10 miles. On portland cement concrete roads we determine a ride score on the outer lane only, again by using the PCA road meter.

In 1972 we developed preliminary guidelines for use of the pavement evaluation data, as illustrated by Table 2. The work type warrant depends on the pavement condition rating. According to our tentative guidelines, from 0-38 we would seal or blanket, from 39-69 we would perform major maintenance, and at 70 or above we would recommend reconstruction. Also, a high sum of defects or ride score value may determine the work type warrant without regard to the pavement condition rating.

Figure 1gives the number of lane miles and the statewide percentage of asphaltic concrete roadways hat fall into several pavement condition rating categories, based on the 1971-72 survey. Note that 65.7 percent of the roads are in the 0-20 category, 16.6 percent from 21-40, 16.7 percent from 41.70, with only 1 percent in the over 70 category.

Figure 2indicates the percentage of lane miles with a pavement condition rating less than 20. It shows the percentage for each district as of January 1970 and January 1972. Comparing the results of the 2 surveys, 3 of the districts improved, 6 were worse, and 2 were unchanged. Statewide, approximately 35 percent of all the lane miles evaluated had a pavement condition rating above 20 in both surveys. This may mean that we are holding our own in pavement maintenance. Our 1973-74 statewide pavement evaluation survey may shed more light in this area.

Other variables to be considered for their effect on pavement deterioration include ADT (average daily traffic) and rainfall. These factors have not yet been evaluated. To date, our only evaluation of PCC pavement has been to determine a ride score on the outside lane of all PCC pavements statewide. We are in the process of establishing guidelines to determine acceptable limits for this figure. Tentatively, a ride score of 40 is considered satisfactory, a score of over 45 is too high and should be improved, and in between is a judgment area. Figure 3 indicates the number of lane miles and the percentage, statewide, of the PCC roadways that fall into these categories: 93.0 percent are in the 0-40 category, 2.2 percent from 41-45, and 4.8 percent in the over 45 category.

We have made extensive use of this rating system in trying to better manage maintenance resources while other functions in the Department of Transportation have also made limited use of our system.

In my judgment, the maintenance branch will make more and more use of this system. We are presently using the information in making budgeting decisions and plan to use the information much more widely as we build up a data bank and learn more about the condition of the existing system, its rate of deterioration, and the effectiveness of our maintenance effort.

SAFETY

The increasing personal exposure to highway traffic of our state highway employees working on the state highway system is cause for grave concern in California.

There recently has been an increase in the number of injury and fatal accidents suffered by state highway employees while working on or along the traveled way. Between January 1, 1973, and July 1, 1973, four maintenance employees were killed and more than 20 others seriously injured in motorist-related accidents on the state highway system.

The potential for our employees to be struck by passing vehicles is a function of the degree of exposure of the empoyees in their daily work. We are implementing and exploring both short- and long-range plans to reduce exposure.

Immediate steps to reduce exposure have been taken, such as a 50 percent reduction in highway sweeping, expanding the use of shadow trucks as protective barriers, elimination of hand edging of roadside vegetation, and news releases requesting the traveling public to be on the lookout for the men who maintain the state highway system.

Our long-range goal to reduce significantly the exposure of our maintenance employees while they work on the traveled way will influence our thinking when considering pavement rehabilitation.

In California, we expend approximately 230 man-years annually on hand-placed surface maintenance activities, such as dig outs, hand patching and sealing, and crack pouring performed on

Table 2. Preliminary guidelines for use of pavement evaluation data: recommended ranges.

FLEXIBLE PAVEMENT · ASPHALTIC CONCRETE

WORK-TYPE WARRANT	MAXIMUM SUM OF DEFECTS	PREFERRED RANGE			MAXIMUM RIDE SCORE	OTHER VARIABLE CONSIDERATIONS		
		SUM OF DEFECTS	CALCULATED RATING	RIDE SCORE		ADT	RAIN FACTOR	OTHERS
Routine Maintenance *								
Seal Coat	20	0-20	<26	0-35	40			
Thin Blanket	40	14-30	<39	20-50	55			
Major Maintenance	90	40-80	<70	40-70	75			
Improvement Recom.	N.A.	40-∞	>70	70-∞	N.A.			
RIGID PAVEMENT (PCC)								
Not Specified	N.A.	N.A.	N.A.	0-40 (Satisfactory)	>45 (Imp Recom.)			

CODE

- > - MORE THAN
- < - LESS THAN
- ∞ - INFINITY
- NA - NOT APPLICABLE

* Guide lines not proposed: Expenditures at any specific location should be less than $5,000. Jobs over $5,000 are considered Major Maintenance.

Figure 1. Statewide summary of AC pavement condition ratings from 1971-72 survey.

CATEGORY	0 - 20	21 - 40	41 - 70	70 +	TOTAL
Lane Miles	19,473	4,944	4,953	30.2	29,672
%	65.7	16.6	16.7	1.0	100.0

Figure 2. AC pavement condition ratings.

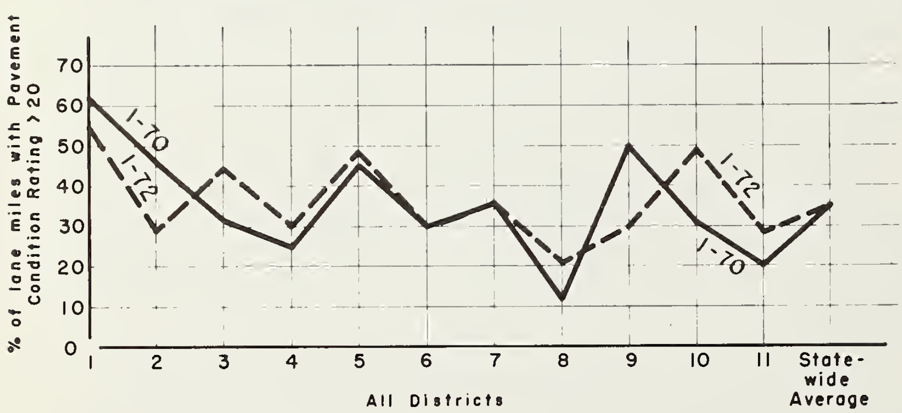


Figure 3. Statewide summary of PCC ride scores from 1971-72 survey.

CATEGORY	0 - 40	41 - 45	45 +	TOTALS
Lane Miles	4,426	107	227	4,760
%	93.0	2.2	4.8	100.0

the asphaltic and concrete traveled way and shoulders. This hand work is high human exposure work and high cost maintenance work. We expend approximately 200 man-years annually on blade and box-placed asphalt mix and seals on the traveled way, which is also high human exposure work.

In the past, warrants for the allocation of resources for rehabilitation of high types of pavements such as construction-financed thin blankets, structural blankets, and major reconstruction have been based on structural inadequacy and high maintenance costs. A new warrant, perhaps called the maintenance employee safety index, may have a major influence in our pavement rehabilitation strategy. If created, this new warrant could tip the scales when decisions are made between continuing to maintain a road with resultant high human exposure or allocating money for more extensive rehabilitation work, thereby reducing employee exposure.

VARIABLES AND CONSTRAINTS

A current estimate is that about $140 million would be required to correct existing highway structural deficiencies in California. Those needs are understated since they do not include roads that also have a capacity deficiency. The money available in 1973-74 to respond to those needs is about $25 million. Obviously, the decision-maker is faced with doing nothing or less than needed on many roads.

Another difficult problem relates to levels of service. How do we equate the quantity or composition of traffic to the resource distribution? Initial construction is fairly clear in regard to geometrics and structural sections. However, maintaining those facilities with a tight dollar constraint is another matter. Even without sophisticated testing equipment, we know which parts of the state have the best roads and which the worst. Should we equalize conditions by emphasizing our effort in the worst areas at the expense of the best, and how does traffic temporize that decision?

These two problems have a large influence on our decisions and the people who make them. We pay premium wages for personnel with the ability to make good decisions. These people welcome advice, but reject rigid standards. For example, most rural area maintenance men use and swear by chip seals. However, most maintenance men in urban areas will not use them because of the bad public relations associated with windshield damage, even though the two roads being considered may be in the same condition and a research study recommends a chip seal. I suspect that each decision-maker is correct in evaluating the use of the repair method for his area. The problem, then, is to devise a system that assists the decision-maker and still leaves room for flexible solutions.

Finally, in the variables and constraints area are the environmental constraints. For example, additional resurfacing of some urban streets is impossible because of center crown and gutter grade controls. In the past the solution has been a piece of equipment called a heater-planer that heats, scarifies, and removes a portion of the existing pavement and allows the placement of new material. Unfortunately, in accomplishing this the machine belches great volumes of flame and smoke as it crawls down the street. Needless to say, this operation has been banned in many parts of the state, and it taxes our sense of morality where we do use it. The point is that traditional solutions to maintenance problems are in jeopardy. Compromise and/or new concepts and approaches are necessary.

How can we remove maintenance and rehabilitation strategies from their major dependence on subjectiv judgment alone and move to a more rational method of predicting rates of deterioration and methods of repair? Do we want a system developed that will lock us into a rigid schedule of maintenance that leaves no room for either subjective judgment or a flexible solution based on existing constraints? I believe we would be more comfortable with a workable system where rational guidelines are provided, along with a reasonable judgment area.

I believe we have a useful tool in our pavement evaluation procedures. However, we do not yet have enough history to prove its value or validity. Pavement evaluation is not a complete picture of the condition of the highway, and the addition of deflection data on a greater scale than present practice may be feasible.

It is my observation that, although we have made some progress in recent years, we have a long way to go to develop systems of pavement evaluation that will place greater reliance on predicted scheduled rehabilitation and less on the individual foreman's judgment on time and method of needed repair.

SPECIAL PROBLEMS WITH AIRFIELD PAVEMENT MAINTENANCE

Douglas I. Hanson

There are millions of square yards of airfield pavement in use in the United States. The Air Force alone owns enough airfield pavement to be able to provide a 200-foot wide runway stretching from the State of Washington to the southern tip of Florida. The problem of maintenance and rehabilitation of this inventory becomes more complex and critical each year. Most of these airfield systems are over 20 years old, with many of the municipal fields built originally during the WPA days or during the build up for World War II. During the late 1950's most of the airfields in the Department of Defense were upgraded to provide airfields for heavy bomber and cargo aircraft of that era. Coupled with these aging pavements is the rapid growth in aircraft traffic and weights. During the period 1958 to 1968 the passenger-miles more than doubled, and it is estimated that the passenger demand will increase more than threefold in the next 10 years. Also, there has been a continuous increase in aircraft size and weight. Aircraft with gross weights of over 3/4 million pounds are now in use and aircraft with a gross weight of 1 1/2 million pounds are expected by some authorities by 1980. Fig. 1 indicates some trends published by the Transport Aircraft Council of the Aerospace Industries Association in transport aircraft size and weight.

ORGANIZATION FOR MAINTENANCE

The airfield network is essentially divided into two large classes: civil airfields and military airfields. Each of these two classes is organized differently for the accomplishment of airfield maintenance and rehabilitation activities. The discussion of civil airfields will be directed toward the operation of the larger and more complex airfield systems such as Los Angeles International or Hartfield International Airport in Atlanta. The discussion of the military airfield maintenance organization will be directed toward those operated by the Air Force because of the larger number and types of aircraft using these fields.

Air Force Organization

The Air Force is comprised of 14 major air commands each with a specific functional area, such as training, communications, and military airlift. Each of these major groups, containing a number of individual Air Force bases, obtains approval and allocation for maintenance and repair funds from Headquarters, United States Air Force. They prepare an estimated budget that is submitted to Hq USAF for submission to the Department of the Defense and to Congress for approval and allocation of the necessary funds.

The Air Force generally tries to maintain at each base an effective pavement maintenance section capable of accomplishing work consistent with the equipment on hand and with the capabilities and limitations of the assigned personnel. Generally, the manpower and equipment are adequate to accomplish the full scope of pavement maintenance, which generally includes repairing concrete spalls and popouts, repairing and resealing joints and random cracks, replacement of random slabs, application of asphalt sand seals, repairing asphaltic concrete potholes, application of single or double bituminous surface courses, replacement of aggregate base courses, and installation of subdrainage systems. To accomplish this work, each base has an airfield maintenance crew that usually is not used for the accomplishment of other tasks such as grass mowing or street repair.

Those activities that require specialized equipment or manpower greater than that available on the base are usually accomplished by contracting to a local construction company. Typical of this type of work is a heater-planer overlay, the replacement of a large number of deteriorated slabs, or the overlay of a large apron.

The work is identified through a number of procedures based on inspections conducted by designated individuals at specific intervals. Once each year a pavement maintenance plan is developed by the engineer responsible for the management of the airfield system. The plan consists of an inventory of all the airfield pavement features (there may be as many as 60 different features in an airfield system), which will include a description of the pavement structure (the thickness of layers and type and strength of each layer), the year in which the feature was built, the design loadings for the feature, the maintenance priority that is established based on the use of the feature and its essentiality to all flying activities, a verbal description of the feature, a condensed history of all major maintenance and repair projects, and any present or proposed maintenance or repair projects (such as joint sealing, overlays, widening, etc.). In addition to this annual plan, representatives from flight operations, flying safety, civil engineering, and the communications organization on the base conduct a survey each month to determine discrepancies and/or irregularities that require repair or that could cause a disruption to flying activities. To accomplish these repair actions, many bases have established runway closures of two 4-hour or one 8-hour period for each active runway for each month.

These closures are coordinated with the flying community so that interruption to the flying schedule is minimized. These closures are usually planned for daylight hours and scheduled for weekends or holidays.

In addition to the yearly plan and the monthly inspection, the operations personnel conduct a daily inspection of the airfield to determine its condition and the presence of any hazards to air traffic. Any problems are immediately brought to the attention of the airfield maintenance personnel for correction.

Civil Airfields

The civil airfields are usually owned and operated by the municipality in which they are located with maintenance funds coming from the airport users in the form of user and concession fees. Examples of this are automobile parking lot fees (up to $4.00 per day) and aircraft landing fees (40 or 50 cents per 1000 lb gross weight). This independent source of revenue means that the airport is self-sufficient and that each individual airport can set its own maintenance policies. The FAA participates in only major rehabilitation and new construction projects and FAA does not set policy on the level of maintenance effort expended by an individual airport. The FAA participates with funding only when the rehabilitation is to upgrade the system for the introduction of new and heavier aircraft or where there has been a structural failure. At those airfields that have not less than 1 percent of the total passenger traffic in the United States, the rate of federal participation may not exceed 50 percent; at those airfields that have less than 1 percent of the total passenger traffic, the rate of federal participation may not exceed 75 percent.

Fig. 2 shows a typical organization chart of a civil airfield maintenance organization. They prepare a budget on a yearly basis by using experience from previous years to develop their anticipated labor and materials costs. In the preparation of this budget, special attention is given to any special repair work that may have to be accomplished by contract. The independence of actions in fiscal matters allows each of the civil airfields to tailor field and terminal maintenance organizations to fit specific problems and needs.

Civil airfields also identify their requirements for maintenance through an active inspection program. They inspect on a daily basis and during these inspections coordinate with the airfield users and the airfield operations section. These inspections are made either once or twice a day by a representative of the airfield maintenance section (usually the field maintenance superintendent), a representative of the airfield lighting section, a representative of operations from the airfield managers office, and a representative of one of the airlines using the field.

The major airfields conduct maintenance and repair activities much the same as those of the Air Force. Because their fields are usually larger than most Air Force bases, they have capabilities to accomplish larger projects such as the repair and replacement of large apron areas or the overlay of a taxiway or runway. They have their own laydown equipment, and some of the airfield maintenance organizations have their own asphalt batch plants.

SPECIAL CONSIDERATIONS

The rehabilitation of airfield pavement systems is designed to accomplish the same objectives as highway pavements: to correct a surface deficiency or structural problem, to improve the skid resistance of the surface, and/or to improve the riding quality of the runway surface. Both airfield and highway pavements are rehabilitated to correct defects such as faulting of slabs, blow-ups, rutting in asphalt pavements, spalling of concrete pavements, and potholes. However, there are a number of problems that are unique to the operation of aircraft from a paved surface. Each of the above three objectives will be discussed to point out aspects unique to the operation of aircraft. Also where possible the proposed or interim solutions to the problems presented will be discussed.

Structural Considerations

The structural design of highway and airfield pavements is similar in that the general principles of pavement design apply to both. The differences between them are the magnitude of loading and the number of repetitions applied to the pavement during the life of the system. The loads are 30 times higher for airfield pavements, and the load repetitions are measured in terms of thousands for airfields versus millions for highways. These differences result in a much thicker structural section for airfields. Many airfield pavements are built with as many as 24 inches of PCC or with a 72-inch thick asphalt pavement structure. Recent advances in multiwheeled aircraft such as the C-5A and the Boeing 747 have further complicated the problem of rehabilitation of airfield pavements.

Studies conducted by the Waterways Experiment Station for the FAA and the Air Force have shown that the operation of these heavier aircraft will cause significant stresses and deflections at much deeper depths than previous aircraft and that these aircraft will cause premature failure of keyed construction joints in concrete pavements supported by low to medium strength materials.

Fig. 3 shows a comparison of the vertical deflection versus depth for a single-wheeled aircraft, the C-5A, and the 747. It can be seen that, at 6 feet, the deflection from these multiwheeled aircraft is 4 to 5 times that of the singlewheeled aircraft. The implication of this is that many of the airfield systems in service are built over fine-grained soils that were of sufficient strength for previous aircraft but are now overloaded. This could mean the failure of these pavement systems due to a structural failure at low depths. A procedure is needed to strengthen these deep layers.

These multiwheeled aircraft also impose a loading pattern on rigid pavements radically different from that previously encountered. The studies at WES indicated that the procedures used for the thickness design of rigid pavements are valid, but they demonstrated that conventional keyed longitudinal construction joints were inadequate for pavements constructed on low-strength subgrades and that the pumping of fine-grained soils through the pavement joints could result in failure of the pavement and pavement joints. As a result of this problem, WES conducted further study for the Air Force and the FAA. That study, recently published, recommended that procedures be developed to strengthen keyed construction joints where the foundation materials are low to medium strength materials (subgrade modulus of less than 400 pci), and that in new construction the use of keyed joints be discouraged for pavements on unstabilized foundations.

There are two other problems of significance in determining the rehabilitation strategy to be used for airfield pavements. These are the jet blast and turbulance effects of aircraft engines and the effects of fuel spillage.

Asphalt is soluble in petroleum products such as jet fuel and hydraulic fluid. The slow evaporating rates of these products have a more severe effect on asphalt pavements than gasoline. In the past, flexible pavements were surfaced with jet-fuel resistant tar-rubber concrete. In recent years, however, this type of pavement surfacing is showing damage from fuel spillage during refueling operations and certain maintenance operations.

Jet blasts and turbulence on a bituminous pavement cause the binder to soften and the pavement to erode. A blast 1,000 feet from the C-5A engines and 75 feet from the center of the aircraft is in excess of 100 mph and presents a problem with shoulder erosion control.

Skid Resistance

The aviation community has become increasingly concerned with aircraft landing safety during inclement weather. This concern has been prompted by the increased landing speeds of jet aircraft and the increased number of wet-weather landings permitted by improved flight instruments and instrumented landing systems. Adequate pavement traction has become a major safety concern to prevent loss of directional control and to provide aircraft stopping ability during landings on wet pavements. During the last 5 years, the number of accidents attributable to hydroplaning has increased with both civil and military aircraft.

The problem of pavement traction within the aviation community has, during the last 4 to 5 years, taken on two aspects: providing a skid resistant surface and removing contaminants from the runway surface.

To provide a skid resistant surface requires first that the condition of the paving surface be assessed to determine its hydroplaning potential. There are two approaches that are being proposed for measuring the skid resistance of runways: the Mu-meter and the diagonally braked vehicle.

The Mu-meter is a small trailer unit designed and manufactured by ML Aviation (Maidenhead, Berks, England) for the specific purpose of evaluating the coefficient of friction (Mu) of runway surfaces. The device physically evaluates the side-slip force between the tires and the pavement surface. It is a continuously recording device that graphically records the coefficient of friction versus distance along the pavement. The friction measuring wheels have tires with 10 psi pressure so that, when the test vehicle is towed at 40 mph, it gives a speed equivalent to 1.2 times the theoretical hydroplaning speed (33mph).

The diagonally braked vehicle is a specially instrumented station wagon used to determine the stopping distance of the vehicle at 60 mph in a locked-wheel mode under a diagonally braking configuration. Instrumentation in the vehicle records velocity and stopping distance.

The Air Force uses both devices concurrently to evaluate the skid characteristics of a runway surface that has been wetted with 0.1 inch of water by an external water source (usually a fire department water tanker).

The results are reported in terms of the coefficient of friction (Mu) as measured by Mu-meter and the wet to dry stopping distance ratio (SDR), as measured by the diagonally braked vehicle. The civil aviation community and the Navy are planning to use the Mu-meter to measure the coefficient of friction. Table 1 gives a rating system that is going to be used by the Air Force to describe the skid resistance characteristics of a runway.

After it has been determined that a potential hydroplaning problem exists it is necessary to correct the problem. The two most promising approaches being used by the aviation community to correct skid resistance problems are a porous friction course overlay and longitudinal grooving of the surface.

The porous friction course has been used on a number of Air Force installations located in

England and Germany, one base in the United States, and a number of commercial airfields. The principle is to allow rapid drainage of the surface water both conventionally over the sloping, exposed top surface to the edges and also laterally through the very porous, open-graded mixture of aggregate bonded together at point contacts with asphaltic concrete binder. This surface can be installed for an estimated $1.00 per square yard.

The other approach is to groove the runway with transverse grooves the entire width of the pavement surface. This is the most popular method with the municipal airfields and has been used extensively in the United States and overseas. This technique was first developed by the British in the early 1960s and tested and evaluated by NASA at the Langley Landing Test Tract at Norfolk, Virginia. They optimized the groove pattern for aircraft operations and recommended that a groove pattern of 1/4 inch deep, 1/4 inch wide and 1 inch center to center be used. Because of economic considerations, other patterns have been used, the most common being 1/4 inch deep, 1/4 inch wide and 2 inches center to center. The most recent projects built for the Air Force using this last groove pattern cost approximately $1.00 per square yard.

With the recently developed measuring equipment and improved surfacing technology, the problem of rehabilitating a runway surface to solve a skid resistance problem is much closer to solution. However, the buildup of contaminants on the surface of the runway, in the form of reverted rubber from aircraft tires, remains a problem.

For the last 3 years, the aviation community has been actively searching for a procedure to remove this rubber from a runway surface. In the last 6 to 12 months a procedure developed by industry has shown great promise. The early projects to remove rubber consisted of using chemicals to dissolve the rubber and then using water tankers to wash the rubber and chemicals off the runway surface. It was soon found that the most effective chemicals were also detrimental to the environment, and so the Air Force decided to try to find another method. A number of other chemicals were tried with varying degress of success. During this period of trial and error with chemicals several companies proposed use of high pressure water. With this procedure, water is applied through nozzles at 7,000 psi for asphalt pavements and 9,000 psi for PCC pavements. The rubber deposit on a typical runway can be cleaned off in 2 working days, and during this period emergency aircraft traffic can use the runway. This procedure is also more cost-effective than chemical procedures. The last few contracts for chemical removal of rubber cost about 7 cents per square foot, whereas using high pressure water has cut the cost to 3 to 4 cents per square foot. The one uncertainty with high pressure water is the effect of continued use on the durability of the pavement surfacing.

Riding Quality

For the past 10 to 12 years, the airfield maintenance engineer has been receiving an increasing number of complaints from the designers and operators of modern high performance aircraft. To the aircraft designer, the elevation profile of the runway is a forcing function that causes dynamic loading of the aircraft during taxi and takeoff modes. Pavement roughness can cause discomfort to the pilot and vibration of the aircraft's instrument panel of such a magnitude that the panel cannot be read. It has been shown by NASA that the roughness of a particular pavement surface will have significantly different effects on different aircraft. Thus, a pavement surface will be source of complaint to the operators of one particular type, but will be considered satisfactory by the operators of another aircraft type. Possible reasons for this are the varying degrees of flexibility of the different aircraft (for example, the B-52 Bomber is more flexible than the F-4 fighter) and the different procedures used during the takeoff maneuver. The approach that has evolved from studies conducted by NASA is to determine the response characteristics of an aircraft during taxi and takeoff modes. Response data for the runway are then compared to an elevation profile of the runway to determine which portions of the runway are causing the problem.

In the initial studies conducted by NASA, actual aircraft were instrumented to determine the response of the aircraft. Accelerometers were placed under the pilot's seat, at the aircraft's center of gravity, and in the tail of several types of aircraft. These studies proved that the approach was feasible, but an easier to use technique was needed. The major problem with using an instrumented aircraft is obtaining an aircraft to do the initial studies and then obtaining the same aircraft after a runway has been corrected. In recent years, work has been done to obtain computer codes for mathematical models of aircraft. Some of this work is now nearing completion. The Air Force has developed a universal code that can be used to model any aircraft operated by the Air Force by changing certain input data. The forcing function for this program is a laser profilometer.

The laser profilometer consists of a laser cart that projects a collimated laser beam down the runway, and a profilograph vehicle equipped with a photoelectric sensor (Fig. 4).

A profile follower wheel, located directly below the sensor assembly, follows the runway profile. This distance between the wheel and the sensor is a direct measure of the elevation of the line being profiled. A direct drive introduces the profile variations between the surface

Figure 1. Gross weight growth.

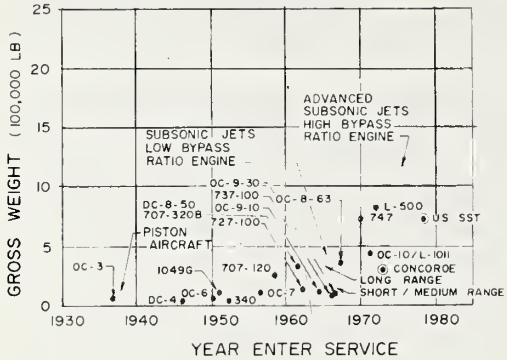


Figure 2. Typical airfield maintenance organization.

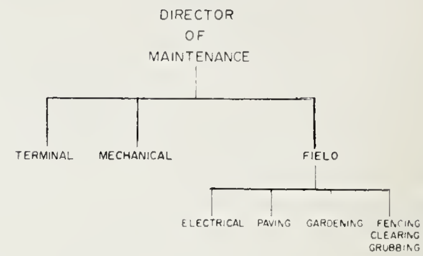


Figure 3. Maximum elastic deflection (in.).

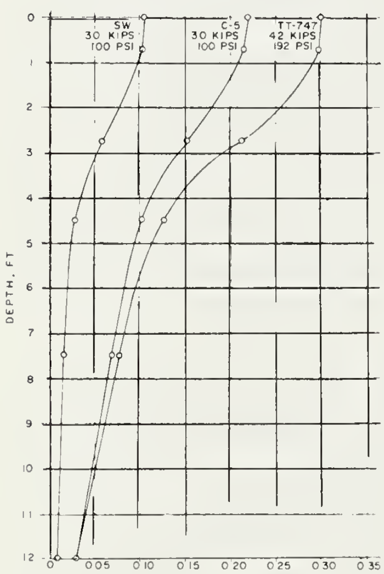


Table 1.

Mu	Expected Aircraft Braking Response	Response
Greater Than .50	Good	No hydroplaning problems are expected
.42 - .50	Fair	Transitional
.25 - .41	Marginal	Potential for hydroplaning for some aircraft exists under certain wet conditions
Less than .25	Unacceptable	Very high probability for most aircraft to hydroplane

Figure 4. Porous friction course.

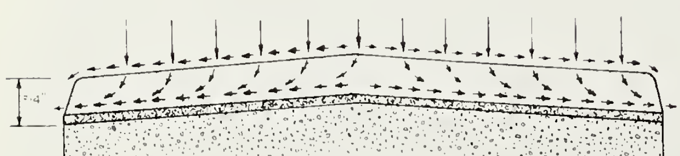
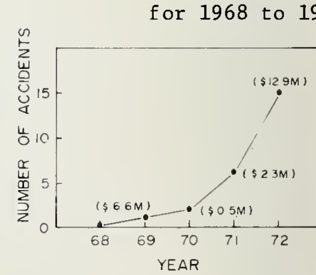


Figure 5. Number of hydroplaning accidents and dollar losses for 1968 to 1972.



and the sensor position, determined by the horizontal reference laser beam, into a digital encoder. At 6-inch intervals along the line being profiled, a switch driven by the profile follower wheel causes data to be read out of encoder and recorded on magnetic tape for data reduction on a computer. In studies conducted to verify accuracy, the system has proved to be accurate within ± 0.01 foot.

The profilometer and computer programming systems are used together to determine the simulated response of aircraft in the taxi and takeoff mode. Fig. 5 shows a schematic of the cycle of operation being used to identify runway roughness problems. It has been determined from physiological studies that, when the acceleration exceeds 0.4 the acceleration of gravity, the runway is too rough.

SUMMARY

Airfield and highway pavement maintenance and rehabilitation activities are accomplished for much the same reasons: correct a structural problem, improve riding quality, and restore the skid resistance of the pavement surfacing. Both types of pavements have some of the same problems such as the rutting of asphalt caused by channelized traffic and spalling caused by poor construction. The major differences between them are that airfield pavement structures are usually thicker than highway pavements, the loads on airfield pavements are much larger, and the number of load applications is lower for airfield pavements.

Most airfield maintenance organizations can accomplish routine maintenance with their own people with major projects being accomplished by contract with local construction companies. Most of these organizations identify their problems through an active airfield inspection program. This means that the airfield manager generally has knowledge of the condition of the airfield system on a daily basis, which is a significant advantage over his counterpart in the highway transportation field.

ACKNOWLEDGMENT

The views expressed are those of the author and do not necessarily reflect those of the Air Force Systems Command, the U. S. Air Force, or the Department of Defense.

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MODIFICATION OF STRUCTURAL DESIGN PROCEDURES FOR PAVEMENT REHABILITATION

REPORT OF GROUP A

W. J. Kenis, Chairman
J. L. Brown, Recorder

Currently available pavement overlay thickness design procedures that were developed through modifications of existing virgin pavement design procedures were discussed as to their classification with the definitions set forth by Witczak. (Virgin pavement thickness design procedures refer to existing pavement thickness design procedures or to the rapidly developing improved or rational design procedures). Two overlay design procedures were examined in detail so that each group member would be familiar with the general nature of such methods. It was determined by the group that the greatest contribution to the workshop would follow from examining the whole class of virgin pavement design procedures that could be modified and used as pavement overlay design procedures. The advantages and disadvantages of using such procedures were enumerated, and a format outlining the essential components of any such modified overlay design procedure was generated. Two specific research needs statements were formulated. In addition, individual statements of research needs were prepared by the group members and are included in this report.

STATE-OF-THE-ART CRITIQUE

Many of the existing overlay design procedures have been summarized by Witczak. It was the concensus of the group not to dwell on the specified techniques that would be encountered through the use of any of these methods nor to expound on the manner in which virgin pavement design procedures could or could not be modified to account for overlay thickness rehabilitation. Instead, the specific advantages and disadvantages associated with the broad range of procedures of overlay design, as they are evolved from virgin pavement design procedures, were discussed in detail.

In most cases existing pavement overlay design procedures fail to recognize the differences in both the primary and limiting behavior that exists between the original and rehabilitated pavement structure. In addition, many of the existing overlay design procedures include only a subjective evaluation of the structural condition of the existing pavement. These procedures use layer equivalency factors that attempt to assess the structural value of the layers in the existing pavement for use in the overlay design procedure. The utilization of equivalency factors such as those embodied in the Portland Cement Association, American Concrete Institute, Asphalt Institute, Federal Aviation Administration, and Corps of Engineers overlay design procedures tends to encourage cursory inspections of the pavement without attempting to evaluate the pavement's primary behavior, limiting distress, causes of distress, and structural condition.

The test time of the performance of existing pavements has enabled the development and verification of many of the existing virgin pavement design procedures. However, there is limited documentation on the performance of overlayed pavements, which has slowed the development and verification of overlay design procedures.

The improvement to any virgin pavement design procedure and its subsequent modification to allow for the design of overlays will depend on the adequacy of pertinent feedback information, i.e., distress and its causes, and on the compatibilities between these parameters and their use in both the virgin and overlay design procedures.

Advantages and Disadvantages

Advantages to modifying a virgin thickness design procedure to accommodate overlay thickness design are as follows:

1. Such modified procedures are more easily implemented because they require less additional training, equipment, and software development than other procedures.
2. Such an approach is compatible with the pavement management system concept because many common inputs and outputs are used. Monitored outputs from one performance period become inputs to rehabilitation design for the succeeding period.
3. Inherent weaknesses in the original design procedure may be exposed when it is modified by design overlays.

Disadvantages to modifying a virgin thickness design procedure to accommodate overlay thickness design are as follows:

1. Weaknesses such as basic assumption errors that are inherent in the virgin design procedure are perpetrated to the modified overlay procedure.
2. Many existing virgin design procedures are empirically based. Extrapolation of such procedures to overlay design is questionable, i.e., whether they can handle new materials or major load changes.
3. Such modified overlay procedures apparently foster the tendency not to evaluate objectively the in situ structural condition of the original pavement before designing the overlay.

<u>Components of an Overlay Design Procedure</u>

The essential components of an overlay design procedure are

1. Condition surveys,
2. Serviceability assessment(s),
3. Structural evaluation to be compatible with basic design procedure input,
4. Evaluation of layer parameters, i.e., layer strength or layer equivalency factors, and
5. Evaluation of remaining life.

PRIORITY RESEARCH NEEDS

Performance data on overlaid pavements are needed to evaluate and adjust any proposed overlay design models as well as to suggest different or new models. Methods must be developed to express the existing pavement structural condition as determined by rapid, objective measurements on the in situ pavement in terms acceptable as input to the virgin pavement design methodology

ADDITIONAL RESEARCH NEEDS

1. The necessary research should be done to develop a complete pavement management system. This should include original design procedures, projected life, costs, anticipated rehabilitation, and maintenance. It should also include a process of reevaluation and assessment and the capability of adjustment and redirection. The system should be all-inclusive with such features as design procedures materials and strength information, condition surveys, traffic, history, costs, trade-offs, etc. The primary benefit would be to management in making decisions on how and when to spend available moneys.

2. The need exists for a small qualified group to make detailed appraisal and evaluation of overlay design procedures. It should include design details, fundamental considerations, compatibility with construction and maintenance procedures, strength and weakness of input and output, and long-time evaluation capabilities. The result should make recommendations on direction for future or more successful design procedures.

3. Performance data on overlaid pavements are needed to test, evaluate, and adjust any proposed models and to suggest different or new models. The need is for all levels of sophistication in such data from gross general data such as the average life of 1-inch overlays to precise data characterizing the condition of the original pavement, the overlay, its response to load, its behavior, distress, and performance.

4. New and innovative rehabilitation techniques and materials are needed. To use them advantageously requires an analysis procedure that might include model tests, field tests, and accelerated loading on which a decision to build such new solutions can be made.

5. There is a need for primary data on pavement performance. These data must come from observations of real pavements in use or from major full-sized test facilities with accelerated traffic. Implementation of this need will involve adequate data banks or selected sections of new and overlaid pavements to provide information on distress, performance, traffic history, and relations.

6. A need exists to ascertain those characteristics of the virgin pavement design methodology that must be present to enable its use as an overlay design procedure, recognizing that an overlaid pavement may behave quite differently from a virgin pavement.

7. A desirable characteristic of a modified existing design procedure would be to incorporate a performance model based on overlay performance studies. The purpose is to estimate the remaining life of the existing pavement and predict the future service of the overlay.

8. A rational method for recognizing overlay design in the theoretical scheme of the existing model is needed.

9. Within the airfield pavement rehabilitation context, where detours and traffic control measures are often not possible, developments in advancing the techniques for measuring in situ layer strength in terms of a cost-effective or rehabilitative design procedure must emphasize rapidity (or at least non-irritability to users) and non-destructability, mostly with an eye to reducing down-time, and user costs due to such down-time. Objectivity is a necessity in reducing human error in data collection and in assuring uniformity in a continuing program of strength determinations that may well be an integral part of long-term performance and serviceability assessments.

10. Present methods of determining airfield pavement serviceability range from zero to archaic. User-based serviceability concepts are only recently being approached from a skid and roughness basis and have not been defined on the basis of such potential parameters as rideability and fatigue of airframe members. There may be several levels of acceptability within airfield areas much as there may be differences in acceptable levels of roughness between primary and secondary highway pavements. Civil and military airfields may have widely differing levels of acceptability, but there has been no definition of these differences, nor have there been definite criteria developed on which to base such differences.

11. A need exists to evaluate and compare several structural design procedures for overlays under a set of given conditions and then to determine from performance evaluation of those in service which procedure is closest to performance, i.e., a test track for control and a comprehensive field measurement and laboratory measurement program so that performance can be determined with repetitive loading.

12. Structural or layer equivalencies for overlay purposes are needed as are objective techniques for determining these equivalencies such as laboratory testing; field testing, both rapid and nondestructive; and connection between lab tests and field tests.

13. Objective techniques for determining remaining life of existing pavement structure are needed and should include assessments of environmental effects and future traffic and decisions concerning the design life of an overlay.

14. A need exists for statistical research of all existing pavement. This is done through a mass road inventory. The data should include condition survey data, performance data (serviceability and riding index), structural data (deflection), pavement data (age of pavement, slip length, thickness, firmness), year of construction, data on traffic, data on drainage, and climatic data. The roads should be divided into permanent sections easily located and marked on the road. A long-term inventory program should be set up. A data bank should be installed. An analysis program (statistical approach) should be initiated.

15. Nondestructive layer strength evaluation needs to be researched.

16. Surface requirements for various conditions need to be defined.

17. Feedback systems need to be instituted to determine inadequacies of existing structural design procedures. Design procedures are modified presumably to improve the procedure. However, we cannot improve the procedure if we do not know what is wrong with it. Feedback systems will identify prevalent failure modes and causes, and then modifications to the design procedure can be devised that will more adequately handle these problems.

18. The methodology for more fundamental design procedures has been developed but is still largely used in a research context and not in practical design. Simplification and standardization are needed to get this methodology into practical use.

LAYERED ELASTIC SYSTEMS

REPORT OF GROUP B

M. W. Witczak, Chairman
L. E. Santucci, Recorder

The major objective of Group B was to consider the use of elastic layered theory in highway and airfield pavement rehabilitation. However, recognition was given to the existence of finite element theory and viscoelastic theory, which might also be used.

The inability of elastic theory to account for nonlinearity or stress dependency of some paving materials was considered to be a major limitation. In particular, consideration of horizontal layer stress dependency might account for difficulties encountered in calculating the shape of deflection basins.

Another major limitation is the inability of elastic theory to deal with layer discontinuities, such as cracks, which obviously violate the assumptions of layered theory with jointed, semirigid or cracked pavements.

STATE OF THE ART

In discussing the applicability of layered elastic theory for use in pavement rehabilitation, the committee considered the theory itself, materials characterization, and failure criteria.

Only a few procedures currently use elastic layered theory in pavement rehabilitation problems. One is that of the Asphalt Institute, which uses elastic theory to compute the reduction in deflection obtained by the addition of an asphalt concrete overlay. (See McComb and Labra). A procedure proposed by Materials Research and Development, (MR&D) provides a rather comprehensive method of evaluation and overlay design, primarily applicable to airfield pavements. It is based on the use of failure criteria, expressed in terms of critical stress or strain, developed from a study of existing conditions of the actual pavement under consideration (see Witczak).

Several procedures measure pavement deflections and adjust the layered theory moduli so that predicted and observed results agree. Initial input layered modulus values may be obtained by laboratory tests, in situ nondestructive wave velocity techniques or assumptions based on existing values from the literature. Various deflection measuring devices, such as the Benkelman beam and road Rater, are used in these procedures. The selection of the deflection device is often dictated by the failure criteria available to the user. For example, if the failure criterion is based on deflections obtained by Benkelman beam tests, this test should be used to measure the existing pavement deflection unless good correlations with other devices have been developed. In the procedure recommended by MR&D, which uses computed stresses and strains as failure indicators, the selection of the deflection measuring device may not be so critical. The types of deflections measured are surface deflection (MR&D), layer interface deflections (South Africa), and surface deflection at various horizontal offsets, i.e., deflection basin approach (Utah, Canada-Anderson). The committee's opinion was that the best potential procedure would be one based on the deflection basin approach because of its ability to measure both deflection and pavement curvature.

Two basic approaches using layered theory are available for defining the broad types of failure criteria. They are the deflection approach and the procedure based on computed stresses and/or strains. The committee felt no general agreement has been reached on which procedure is preferred.

Failure criteria using stress and/or strain can currently be grouped as criteria developed in place from a layered analysis of pavement areas having variable degrees of distress such as used in the MR&D approach; criteria developed from lab tests; and provisional criteria developed from analysis of exsiting empirical design procedures, such as those of the Asphalt Institute, Shell, Kentucky Highway Department, and so forth.

After discussing the state of the art on relating measurable pavement parameters, such as deflection and curvature to theoretical stresses or strains, the committee generally agreed that curvature is a comparative indicator of tensile strain at the bottom of a bound layer and that both parameters are subsequent indicators of pavement cracking.

The relationship of deflection to strain and subsequent distress type was not generally agreed upon by the committee. It was noted that the vertical compressive subgrade strain is viewed by some as an indicator of permanent deformation. This distress may be viewed as a transverse deformation. The concept of pavement roughness, however, is primarily a longitudinal deformation difference phenomenon. This distress may be due to not only stochastic load associated deformations but also non-load-associated causes, such as material and construction variability, differential frost heaves, moisture conditions, etc. However, if one uses the provisional vertical strain concepts proposed by Shell and based on the results of the AASHO Road Tests, the vertical strain criteria reflect not only pavement deformation but roughness as well.

It was noted that different interpretations of distress due to roughness are associated with highways and airfields. In highways there is a tendency for the short wavelength to be more critical to roughness, while in airfields (runways) recent research has shown that long wavelengths are more critical on pilot control, passenger comfort, and dynamic aircraft-pavement effects.

MAJOR FACTORS CONSIDERED

Based on our discussion of the state of the art of elastic layered systems for pavement rehabilitation, 7 major factors or potential research areas were selected and discussed by the committee. These items are shown in Table 1 with a code statement relative to the state of the art knowledge, their implementable position, and a general research needs priority for each.

Applicability to New or Nondistressed Pavements

The ratings shown in Table 1 are applicable only to the elastic layered system for use in pavement rehabilitation. It was the consensus that the state of the art of the theory itself is well advanced and is currently implementable in pavement design for structures not having any joints, such as asphalt concrete (AC) pavements.

Applicability to Cracked Pavements

One of the main limitations of this theory is the current lack of technical information on whether the use of a reduced modulus to account for crack discontinuities will yield adequate correlations to measured stresses, strains, or deflections.

Materials Characterization

The committee agreed that a wealth of material characterization data (E modulus) is available. However, it was also felt that the greatest research effort needed is to verify lab characterizations with actual field values.

Failure Criteria

This factor was viewed by the committee as one that deserves high research priority. Included in the scope of this item is a delineation of what the failure parameter(s) should be in a future implementable elastic layered-based rehabilitation scheme, how to establish the magnitude of these failure parameters, and what type of distress they measure.

Remaining Life Concept

The committee felt that much research work is needed to better define the methods of determining the remaining life of a pavement system. However, it should be realized that this concept may not be needed for many failed pavements. Because most pavements to be overlaid fall in this category, an unduly high priority research need may not be warranted.

Applicability to Performance Model

This factor is probably the most critical item needing considerable research and one that has a relatively poor state of the art.

Current Implementable Procedures

This factor refers to the current availability of an implementable elastic layered method for structural pavement rehabilitation. It is the committee's opinion that reasonable progress has been made to date. Some implementable procedures are present, but they could be improved with the future research proposed in the above-listed research areas.

Table 1. Elastic layered systems research needs.

Item	State of the Art[a]	Implement-ability[b]	Research Need[c]
Applicability of layered elastic theory			
New or nondistressed pavements	5	Yes[d]	4
		NR[e]	4
Cracked pavements	2	NR	2
Materials characterization	4	NR	2
Failure criteria	3	NR	1
Remaining life concept	3	No	3
Applicability to performance model	2	No	1
Current implementable procedure	3	NR	1

[a]1 = poor; 5 = excellent.
[b]Yes = implementable now; NR = needs additional research; No = needs considerable research.
[c]1 = highest priority; 5 = low priority.
[d]For flexible, semirigid, and CRC pavements.
[e]For jointed portland cement concrete pavement.

DEFLECTION, CURVATURE, AND STIFFNESS BASED PROCEDURES

Report of Group C

W. M. Moore, Chairman
D. E. Peterson, Recorder

The members of the group identified and reviewed 10 overlay design procedures based on deflection, curvature, or stiffness.

Table 2 identifies the general characteristics of each and the procedures are listed below:

1. California--George Sherman
2. Utah--Dale Peterson
3. Oklahoma--Dale Peterson
4. Road and Transport Association of Canada (RTAC)--Fred D. Young
5. Transportation Road Research Laboratory(TRRL)--H. W. Lister
6. Virginia--C. S. Hughes
7. Texas--W. M. Moore
8. Pennsylvania--Wade Gramling
9. The Asphalt Institute (TAI)--John Huffman
10. South Africa--Malcom Grant

After we reviewed the procedures it became apparent that there were common threads between them. For example the following seemed generally evident:

1. Deflection data are not used to determine whether an overlay is needed. The warrants for needed overlays are established by pavement surface distress or a change in mission involving heavier or more frequent loads.
2. High deflections were thought to indicate more rapid deterioration of a pavement due to traffic.
3. Bonded pavement overlays will result in reductions in deflection and thus increase remaining pavement life.
4. Most of the procedures provide a means of estimating remaining pavement life based on some traffic and deflection relationship.
5. Most of the procedures are used to determine flexible pavement overlays.

Although there were a number of common threads among the various procedures, it was apparent to the group that none of the systems met a criterion for universal application. In general, none of the systems appeared to be interchangeable because of differences in their relationships between deflection and performance. It was concluded that these differences must be attributable to variations in materials, environment, and measuring techniques. There was limited experience in adopting deflection overlay procedures developed by others. Caution should be used by anyone attempting to adopt any of the procedures especially in regard to these factors.

It was not possible for the group to subjectively rate the 10 procedures. It was the consensus of the group that each one was meeting the needs of the area for which it was developed.

THRUST OF CURRENT RESEARCH

Most of the procedures have been in use for a limited period of time, and the users believe that feedback data are desirable to validate and improve procedures by including such factors as material differences and environment.

IMPLEMENTABLE ITEMS

Work is required to resolve the differences in the various deflection performance relationships, which should include materials and environmental differences. Most of the deflection overlay design procedures could be implemented in another location provided the deflection performance relationships are resolved.

The characteristics of a desirable deflection overlay design procedure were discussed and the qualities thought most significant are as follows:

Table 2. Characteristics of 10 overlay design procedures.

Method	Input	Assumptions	Processing Data	Output	Advantages	Disadvantages	Remaining Life
California	Deflection measured by traveling deflectometer or converted from Dynaflect or road rater reading by appropriate conversion factor	Measured deflections are representative of poor conditions; deflections can be reduced by additional layers; sections with deflections greater than 80th percentile will be corrected before overlay	Data recorded on chart; sections of similar deflections are grouped and data are processed through computer to determine 80th percentile deflection	Recommended thickness of overlay: alternate procedures used such as for reconstruction of truck lanes	Use of equipment (deflectometer, road rater, Dynaflect) providing rapid nondestructive testing		Assumed when overlay is requested that it will be for some design traffic load; deflection obtained for existing thickness of pavement automatically allows for remaining life; the lower measured deflection, the thinner the overlay to carry traffic load
Utah	Deflection measured by Dynaflect with 5 sensors; traffic loads with estimated percentage increase; condition survey; materials costs	AASHO eqs. relating deflections to loads are applicable; fall deflections are proper; sampling and testing errors are compensated for; temperature during testing not critical; age of old pavement not a factor	Determine deflection parameters, 80th percentile, standard deviation, avg. coefficient of variation, SCI, and BCI. Condition statement based on deflection basin parameters; evaluate surface distress. Determine remaining life and traffic data, required deflections from nomograph, structural number for overlay from nomograph; select overlay type and thickness	Recommended thickness of overlay, design life, and material type	Rapid, nondestructive, and simple	Cannot be easily applied to new materials; models need to be verified	Use measured deflection and AASHO modified eqs. for 18-kip axle loads to failure; determine remaining life in years by using present traffic and percentage increase
Oklahoma	Benkelman beam deflections; detailed condition survey	Traffic levels do not have an effect and deflections less than 0.022 in. are satisfactory	Avg. deflections corrected for seasonal variation and for similar areas; nomograph deflection vs. thickness	Asphaltic concrete overlay in inches	Rapid, nondestructive, and simple	Models need to be verified; developed for 1990 needs and classification study	Life curves using years since improvement and percentage depreciation from condition survey
RTAC	Benkelman beam deflections (for design); RCI; condition survey; maintenance costs (for decision-making); traffic	Section uniformity exists; deflections are measured at the proper time	Calculation of $\bar{x} + 2$ from field deflection measurements; selection of limiting deflection	Inches of gravel base	Relatively simple	Field data collection	Not estimated
TRRL	Condition survey; deflectograph deflections and temperature data; estimated required design life (usually 10 years)	Empirical relation exists between deflection and cumulative standard axles; failure criteria are properly defined and overlay will extend pavement life	Data used to define weak areas for repair prior to overlay and to divide pavement into lengths of reasonably constant deflections for overlay design; design for 95 percent confidence using statistical techniques	Thickness of overlay	Rapid and nondestructive, based on results of pavement sections in England over a period of time	Does not apply to very weak or very strong subgrades	Estimated from charts relating remaining life to deflections given knowledge of traffic
Virginia	Decision to overlay (usually because of distress); Dynaflect readings made every 100 ft	Virginia thickness index eqs. are valid for overlays; system is elastic 2 or 3 layer; curves developed through Chevron program	Avg. maximum deflection from Dynaflect with spreadability $(s = \frac{g_1+g_2+g_3+g_4+g_5}{5} \times 100)$ Point on chart is read as in-place thickness index, which is subtracted from thickness index to provide overlay thickness required to bring facility back to original design strength	Thickness of plant mix overlay required; indication of weak subbase, base, or surface	Improvement over guess or standard thickness procedure; can indicate probable weak areas	Limited experience of 2 to 3 years	Only considered as a function of original design
Texas	Dynaflect data, traffic data, temperature coefficient, non-load-associated deterioration parameters, cost data, and constraints imposed by designer	Empirical relation exists between surface curvature index and load associated deterioration; among subgrade soil, environment, and non-load-associated deterioration; and among thickness of overlay, stiffness, existing deflection basin, and deflection basin after overlay	Procedure is computerized including analysis of data to get uniform sections of SCI for design based on ANOVA	Several alternate overlay design strategies ranked in ascending order of total estimated cost discounted to present worth	Comprehensive system for planning including most significant factors that affect total overall costs	Empirical models need improvement	Empirical deflection vs. traffic performance equation
Pennsylvania	Road rater deflections (some Dynaflect) with 11 readings at 100-ft spacing for representative 1,000 ft mile; pavement temperature, season; traffic count and weight	Curvature can be used to identify pavement structure condition; characteristic deflection for design is $D_c = D + 1.65\sigma$ or 95th percentile level; tempera- is corrected to spring; deflections allowed and are related to overlay thickness; AASHO data can be used for permissible deflection	Manual determination of overlay using tables is possible; computer solutions using road rater input is in practice	Design overlay tabulation for total 18-kip EWL life vs. thickness of overlay	Fairly rapid; gives planning selection in overlay depth vs. life; viable procedure considering present life		Deflections related to remaining 18-kip EWL from AASHO Road Test
TAI	Rebound deflections; pavement temperature; traffic (DTN)	The higher the pavement deflextion, the shorter the time and more need for overlay; tolerable deflection is function of traffic; additional thickness of asphalt concrete on existing pavement reduces deflection to an acceptable level	Mean pavement temperature computed; recorded rebound deflections used to determine representative deflections ($\bar{x} + 2$), which are then adjusted to standard 70 F. Conversion of projected traffic to DTN for thickness determination	Overlay thickness: inches of asphaltic concrete	Allows use of pavement response to determine additional structural thickness to reduce deflection	Present technique uses Benkelman beam; thus limited measurements available for a period of time	Through deflection measurement and traffic analysis
South Africa	Need established by condition survey, serviceability, and maintenance costs; deflections measured with LaCroix deflectograph; areas not normal for section are identified for special consideration; traffic data	Failure mode of overlaid system is same as original system, i.e., predominantly deformation oriented; failure by fatigue or asphalt cracking alone will tend to invalidate assumption; design deflection estimated from future traffic for design life of 5 years	95 percent level of deflections calculated for uniform sections; compared with TAI and TRRL procedures in terms of deflection life used to help determine mechanism of failure; limit deflection criteria used; regression analysis made between condition and deflection; deflection life curve from previous correlations; thickness is h(mm) = 277 log (95th percentile deflection design deflection)	Thickness of overlay in mm of asphaltic concrete	Criteria derived for local climate, conditions, and materials; independent of time and can be conducted any time of year	Cannot be readily used by others	Through use of deflection life curve

Characteristic	Quality
Input	Materials Thickness New material properties Climatic conditions Deflection basins Condition survey - distress Traffic Design period
Processing	Theoretical or empirical approach Confidence level Corrections for temperature Remaining life Locate problem areas Data analysis
Output	Thickness Predicted life

RESEARCH NEEDS

Research Needs in relation to overlay design are ranked in order of highest priority.

1. Fundamental deflection basin relationship to performance and material properties,
2. Seasonal variations in deflections for adjustment to critical period (possibly spring conditions),
3. Material characterization from laboratory to field,
4. Use of deflections for material types such as flexible, rigid, composite, etc.,
5. Initiation by FHWA of a demonstration projects program to promote better understanding and use of overlay design procedures, and
6. Relationship of deflection to vehicle speed.

RELEVANCY STATEMENT

An overlay design procedure is most relevant to any rehabilitation system. It appears that a deflection-based procedure would offer an economical method of approach that could be refined conveniently with a data feedback system.

OTHER PROCEDURES

Report of Group D

R. V. LeClerc, Chairman
L. M. Womack, Recorder

Although several scientific methods of thickness design for pavement rehabilitation overlays have been developed and implemented, most current overlay design practices depend heavily on experience and/or judgment of the designer. This was the consensus reached by Group D after much deliberation. A decision was made to treat this traditional method as the principal other procedure, to examine critically its capabilities and limitations, and thereby to have it serve as a basis to which the more theoretical, rational, or sophisticated procedures might be compared.

Elements considered in the group's examination are described below.

PAVEMENT REHABILITATION REQUIREMENTS

A condition rating that includes structural and rideability considerations, subjective inspection, and poor skid resistance (high accident rate) was cited as the pavement distress condition that most often resulted in a pavement rehabilitation program. Other influencing factors were defined as public or political request, a change in traffic pattern and/or loading, and cases where inventory data or advance programming have forecast a need.

Structural improvement, restored rideability, and increased road safety (thru improved skid resistance) are principal reasons for pavement rehabilitation. Each bears a close relationship to the others in that improvement in one area generally results in some improvement in the other areas. Additional purposes were discussed, but the group concluded that the three reasons cited were predominant.

DESIGN THICKNESS

The primary purpose of the rehabilitation program must be identified before the pavement thickness can be determined by using traditional procedures. Once the purpose has been established, a range of thickness may be selected depending on which traditional procedure is used. In the case of structural improvement, this range is generally 3 to 6 inches and can be placed in 1 to 3 lifts. Greater thickness is used in cases where greater traffic intensities or loadings are a strong design consideration. Rideability is restored with thicknesses ranging from 0.5 to 1.25 inches. Portland cement concrete and asphaltic concrete pavements are given separate consideration when improvement of skid resistance is the primary factor for rehabilitation. A 0.5 to 0.75-inch thickness of asphalt concrete, or a seal coat, was determined to be the traditional design for asphalt pavements. Designs for PCC pavements were not well standardized or defined by the working group. A range of practices from seal coats to 4-inch thick asphaltic overlays were reported; however, 1.5 to 3.0-inch overlays were considered as most typical. Within each of the traditional methods cited, it was determined that other considerations were given weight in arriving at a final design thickness. Average daily traffic, the condition rating and its rate of change, and road restrictions are typical.

ADVANTAGES OF THESE PROCEDURES

For each traditional design procedure, a number of advantages were recognized. Simplicity, low engineering and administrative costs, and minimum lead time required to obtain a design value are those most easily identified. One additional advantage cited was the immediate implementation of the method by another user. These traditional methods can be adopted by another user with virtually no modification and put to immediate use in overlay design without having to establish the validity of the method. Versatile and Non-theoretical are terms used to describe other, lesser advantages.

DISADVANTAGES OF THESE PROCEDURES

Dependence on individual experience and background rated high as one of the primary disadvantages of these procedures. Of equal status was the inability to extrapolate the design to new conditions or situations, e.g., extending the traditional method based on local light-duty road experience to the design of an overlay for an Interstate highway. Lack of definition of the judgment factors going into the design considerations and the resulting lack of design uniformity can lead to misunderstanding within management and administrative functions. Finally, these methods do not readily provide for consideration of cost effectiveness.

RELIABILITY OF THESE PROCEDURES

In general, reliability depends on the experience and judgment of the designer. It is also true that reliability is generally good because the traditional design procedure has as its foundation the actual pavement performance for many design situations. As discussed previously, reliability may be greatly reduced when the procedure is applied to new situations. In some instances, the reliability of the design may be affected by internal or external politics.

IMPROVING PROCEDURES

Conclusively apparent in the discussions on this subject was the need for adequate record keeping. Availability of data describing the present composition of the roadway structural section and the maintenance history will increase the designer's capability and effectiveness in selecting proper overlay thickness by exercise of judgment. Likewise, judgmental elements considered in the design should be documented for reference in future evaluation by performance records.

These needs can be met by a data bank that incorporates those features broadly described as pavement and condition inventories plus design documentation. Such a data bank, properly designed with appropriate features for updating and retrieval, will have application to any design procedure. Its features should include compatibility with other elements or subsystems of a more inclusive pavement management system.

RESEARCH NEED STATEMENT

Define and design a data bank model to incorporate necessary elements of overlay design, evaluation, and control plus compatibility with total needs of a pavement management system, including ready retrieval capabilities.

CONCLUSIONS AND RECOMMENDATIONS

Whereas we recognize scientific shortcomings of the traditional method of designing overlays by judgment and experience, it was the consensus of the group that this method would still find considerable application before more rational methods are fully accepted and implemented. It was also the feeling of the group that more refined methods are desirable, and their development should be vigorously pursued.

In addition, it was concluded that the shortcomings of the traditional method are related not so much to deficiencies in serviceability as they are to difficulties in optimizing and administering a cost effective rehabilitation program. Perhaps the success accruing to this judgment and experience method stems from an uncounscious application of the Bayesian theory.

Two other problems facing practitioners of overlay design were recognized in discussions. Although not strictly within the province of the group, or possibly not even of the workshop, they were nevertheless considered very real and present. They are mentioned here as needs, primarily to call attention to their existence so some thought might be given to their solution.

1. A need for more economical means of maintaining skid resistance on PCC pavements throughout structural life of the pavement. Present methods are either short-lived or expensive. The situation is aggravated when remaining structural life is significantly high at the time the overlay is called for.

2. A need for more economical means of rehabilitating structurally distressed travel lanes of multilane roadways where passing lanes show no evidence of distress. Many miles of pavement constructed during the early part of the Interstate program are in this condition. Conventionally constructed overlays result in thick sections to accommodate the distressed lane, where it is needed, and equally thick sections over the passing lane, where it is not needed. Additional material must also be placed on shoulders solely for maintaining a satisfactory cross section. Cost effectiveness of such rehabilitation procedures is questionable, making this problem a worthy subject for analysis.

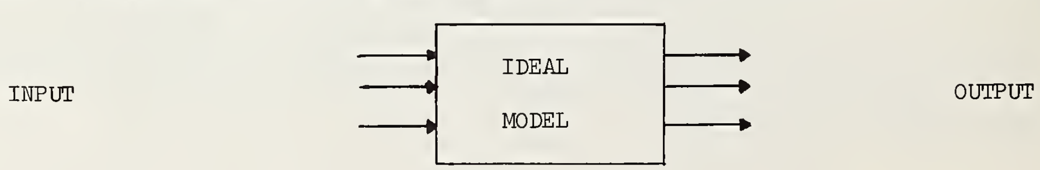
In conclusion, it is believed that economic solution of the problems described above, plus an appropriate record system for control of overlay design, construction, and evaluation, will increase effectiveness of designer efforts in pavement rehabilitation.

IDEAL PROCEDURES

REPORT OF GROUP E

D. I. Hanson, Chairman
Moreland Herrin, Recorder

To develop an improved procedure for the design and construction of pavement rehabilitation projects requires that the ultimate goals of the research effort be defined. Group E was concerned with criteria for the ideal model to predict rehabilitation actions and limited its considerations to the ideal criteria for input and output. We did not consider the following possible inputs and outputs to the model: economical decisions (i.e., interest rates), constraints on the available materials, and overriding control factors such as political social contraints. A shcematic of the visualized model is shown below.



IDEAL CRITERIA FOR INPUT INTO A PAVEMENT REHABILITATION MODEL

Input data are dictated by the needs of the model and can be considered in two broad categories: data necessary for monitoring for strategy decisions such as planning, programming, timing of actions, and network coverage and data necessary for predicting failures, material behavior, and thickness design.

Characteristics of Input Data for Strategy Decision

The committee outlined certain characteristics for input data that would be needed to make the decision of when and where pavement rehabilitation action is needed. The following is a list of characteristics that were thought necessary in the ultimate development of a pavement rehabilitation system:

1. There needs to be a provision for measuring those factors related to the serviceability of the pavement system such as deflection, roughness, skid resistance, etc.

2. Input data should be collected on a continual basis. Data should be collected on every point within the pavement system, and this data bank should be sampled for analysis, rather than collecting the data based on a sampling procedure.

3. All data should be collected simultaneously for purposes of correlating different symptoms of pavenent performance with each other.

4. The data should be stored in usable, manageable forms. The data should be stored on tapes, micro photographs, etc., so that they are accessible. Procedures need to be developed for fast retrieval of the data so that they can be quickly and easily displayed.

5. The data collection system should cause little disruption to the road user and non-user of the pavement system.

Characteristics of the Input Data

The following characteristics were listed as desirable for use in predicting pavement failures and material behavior and developing the required thickness:

1. Continuous,
2. Simultaneous,
3. Easily stored,
4. Nondisruptive to the road user and residents adjacent to the road,
5. Nondisruptive to the behavior of the pavement materials (cells embedded in a granular layer create discontinuities),
6. Nondestructive data collection,
7. Collection of certain historical data built into a data bank: environmental temperature, rainfall, traffic (volume and type of vehicles), maintenance, and repair data (type of work, materials used, costs).

IDEAL CRITERIA FOR THE MODEL

The group spent very little time on what the ideal characteristics of the model should be, but the following criteria were discussed:

1. The model should consider fundamental properties of the materials (moduli, fracture).

2. The model should have the ability to analyze the effects of varying inputs (such as different temperatures, nonhomogeneous materials properties).

3. The model should in the analysis consider the stocastic nature of the reliability of the input data and the variations produced during construction, and

4. The model should have the ability to produce all of the outputs listed in the next Section.

IDEAL CRITERIA FOR OUTPUT

The primary purpose of the model is to predict. Thus, most of the output should forecast the results and events that will occur under a set of input conditions. The output then should reflect the capabilities of the model and thus what is ideal output must be developed within the model. Not only are the following factors ideal output data, but also they are necessary characteristics of the model.

1. Predicts all forms (types) of failures: location, type (shear, fracture), degree or measure, depth of rut, amount of cracking, and when the failure will occur;

2. Predicts changes in performance (serviceability) as evidenced by skid resistance, rutting, and roughness by specifying when, where, and to what degree;

3. Predicts changes in the characteristics of the structural components of the pavement structure, both physical and environmental, (strength, shape, moisture);

4. Predicts whether frost action or damage from severe cold climates will occur and the consequential influence on serviceability;

5. Predicts the outcome of the interaction between forecast distress and performance (i.e., what are the changes occurring in all serviceability factors when one type of distress is predicted);

6. Predicts all costs related to the strategies and thereby facilitates prediction of the various courses of action for use in the strategy decision; and

7. Monitored serviceability data are checked against the forecast conditions to determine whether there is a significant difference between forecast and actual conditions.

RESEARCH NEEDS

The following is a list of those areas the committee felt needed research to develop the ideal system. The list is grouped into 2 categories, those associated with a monitoring system and those associated with a predictive subsystem. All of the research needs were evaluated with the idea of establishing research priorities.

Monitoring System

1. Develop equipment and data collection systems that will enable an agency on a concurrent, simultaneous, and rapid basis to monitor skid resistance, roughness, surface condition, and structural adequacy in a nondisruptive, nondestructive manner (No. 2 priority).

2. Develop nondestructive continuous recording equipment for measuring traffic type and load.

3. Develop a system for continual storage update and retrieval of maintenance and repair history (including costs) and construction information with the quality control data.

4. Develop techniques for inputing accident data into the decision strategy.

5. Develop procedures for incorporating environmental factors into the system.

6. Develop a system for evaluating the effect of changing the ecological characteristics of the area around the pavement system on the life of the pavement system.

7. Develop statistical procedures for determining the timing and locations of sampling of data for model input.

8. Include in the monitoring system equipment for the continuous monitoring of noise.

Predictive Models

1. Develop a model or models that can be used to predict performance, including a capability to (a) predict rutting, cracking, and disintegration from both load and environmental factors (No.1 priority); (b) predict increase in roughness levels; (c) predict loss of skid resistance; (d) predict changes in the fundamental characteristics of the component materials in a pavement structure

over time (i.e., strength, thickness, durability, moisture, No. 3 priority); (2) predict frost problems and effects of severe cold climates (permafrost, frost heave, and spring breakup); and (f) predict user costs.

2. Develop a capability to predict the variability of performance based on variability of input data (stocastic Procedure, No. 4 priority).

3. Develop a capability to correlate distress mechanisms to performance.

CONDITION SURVEYS

REPORT OF GROUP 1

K. H. Dunn, Chairman
D. J. Lambiotte, Recorder

In the paper prepared by Hudson and Finn for this workshop, the diagram shown in Figure 1 was used to relate pavement subsystems to associated monitoring methods.

The workshop assignment included a review of condition surveys as a means of identifying visible pavement distress for the purpose of determining advantages and disadvantages of present survey systems; of identifying problem areas, either in the conduct of surveys or in relating survey results to pavement maintenance or management systems; and of providing a suggested research program to overcome these problems. The term condition survey was defined as any process of identifying, either qualitatively or quantitatively, visible manifestations of pavement distress.

Condition surveys are conducted in a variety of fashions and with varying degrees of accuracy, subjectivity, and reliability by the many agencies employing such surveys. The group noted, however, that most condition surveys were conducted for purposes that were relatively limited in number and that could be briefly summarized as follows:

1. To be used as input to development of a structural rating or index,
2. To aid in projection of budget requirements,
3. To aid in decisions to perform or not perform maintenance,
4. To act as a diagnostic tool for assessment of design and/or construction procedures (certain distress types relate to shear failure, subgrade failures, etc.),
5. To be used as input to rehabilitation design, and
6. To be used as input in determination of pavement performance history.

STATE OF THE ART

Some of the existing survey techniques reviewed by the group included those used by state highway agencies in Minnesota, Washington, North Dakota and the Canadian Good Roads Association and airfield condition survey techniques used by the U. S. Navy and Air Force.

The condition survey techniques currently used by these agencies are quite similar in that the tabulation and subsequent condition rating procedures utilize only subjective measures of pavement distress. (An exception is the survey technique developed by the Navy for airfield pavements which uses an objectively-based distress tabulation system and an associated defect severity weighting procedure.)

Nomenclature used by most agencies in describing pavement distress types is also quite uniform and is categorized by Haas into the following categories:

1. Cracking (longitudinal, transverse, alligator, map, reflection),
2. Disintegration (raveling, stripping, spalling, scaling),
3. Permanent deformation (rutting, faulting, scaling), and
4. Distortion (settlement, heave).

The greatest deviation in survey procedures noted was seen in the weighting factors assigned various distress types by different agencies. In addition to weighting deviations, Group 1 noted other variations in condition survey methods. These are briefly tabulated in Table 3.

PROBLEMS

As a result of these efforts, Group 1 identified the following problem areas:

1. Undesirable subjectivity in surveys due to present techniques and/or human factors.
2. Absence of a valid, workable statistical sampling procedure for highway surveys.
3. Inadequate delineation of established survey areas for repetitive survey purposes.
4. Lack of uniformity in severity weighting techniques for distress types.
5. Inability with current data storage and retrieval methods to achieve a valid and workable inventory of pavement condition.
6. Condition surveys, as currently conducted, are hazardous to survey personnel and disruptive to traffic on urban freeways.

RESEARCH NEEDS

After review of the state of the art for condition surveys and in response to the problem areas outlined above, Group 1 suggests that consideration be given to the following research efforts:

1. Develop uniformity in condition survey techniques and procedures by identifying common survey procedures that may be standardized on a national or regional basis and by developing an implementation program, to include procedural manuals and possible training centers, similar to the FHWA regional skid calibration centers.
2. Develop machanistic/objective techniques and equipment for condition surveys. Suggested areas might include holography (either visual or acoustical), modified radar techniques, infrared sensing techniques, and photographic techniques. An ideal objective for this research area would be the development of a universal testing vehicle that would be capable of conducting pavement skid, roughness, and condition surveys simultaneously and at highway speeds. A suggested acronym is

   R---outine
   O---bjective
   A---utomatic
   D---iagnostic
   S---ystem

3. Define the relationship between distress a d performance. There is a need to develop both pavement condition or distress index and serviceability/performance relationships. The result of these factors may be used in program planning for maintenance and rehabilitation. The work should develop methods for establishing proper weighting functions for combining the various types of distress into a common index and a method of relating this distress index to the PSR or serviceability index.

MEASUREMENT SYSTEMS

REPORT OF GROUP 2

Eldon J. Yoder, Chairman
L. Wade Gramling, Recorder

Group 2's initial deliberations dealt for a short time on discussions of roughness measuring equipment, including profilometers and roadmeters. Group 1 was considering condition surveys and their discussions would probably include methods of measuring pavement condition. As a result of this, Group 2 decided to limit its discussions to methods of measuring properties dealing with structural overlay design. This immediately precluded methods of measuring pavement roughness and pavement profile. The charge of the group as interpreted in the meetings dealt specifically with four items to be considered:

1. Establish an inventory of existing methods for measuring properties pertinent to structural design of overlays.
2. Make a critical review of the properties to be measured by each system and discuss the limitations and advantages of each.

Figure 1. Relation of pavement subsystems to associated monitoring methods.

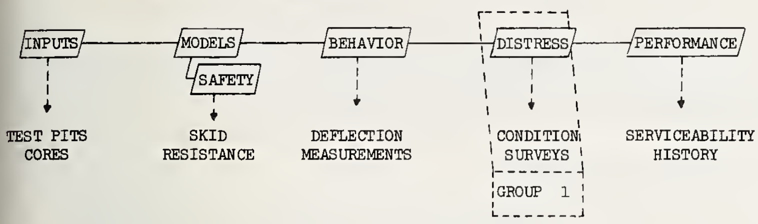


Table 3. Variations in survey methods.

Survey Method	Agency: Highways	Agency: Airfield
Sampling	Various from a subjective selection of sections after an initial review, to statistical selection	Same as for highways
Survey Limits	Total mileage by mile long sections; to existing construction project limits	Facility Limits
Visual vs Physical Measurements	Varies from entirely visual observations; to combination of visual plus physical measurements	Generally visual only. Some physical measurements on flexible pavements
Differentiation Between Rigid & Flexible Pavements	Yes	Yes
Scheduling of Surveys	Varies from conducting surveys on needs basis only; to annual or periodic surveys	Same as for highways
Type of Manpower Used	Generally 2 to 3 technicians, occasionally an engineer as crew chief	Generally an engineer with one or two technicians
Computer data Storage	All have provisions for data storage--most store unprocessed as well as processed data	No data storage

Table 4. Basic types of evaluation devices.

Category of Device	Principle of Operation	Device	Primary Use: Rapid Survey	Primary Use: Diagnostic or Design
A.	Bearing Values	Plate		X
B.	Deflection	Benkleman Beam	X	X
		California Deflectometer	X	X
		LaCroix Deflectometer	X	X
C.	Curvature	So. Africa Curvature		X
D.	Impact	Washington State	X	X
E.	Vibratory	Shell		X
		Road Rater	X	
		Dynaflect	X	
		Cox (under development)	X	
F.	Wave Propagation	Corps Engineers (WES)	X	X
		Air Force (under development)		X

3. Present possible means of improving each system of measurement.
4. Evaluate the adequacy of each system in light of speed of operation and other factors.

Group 2 would not be able in the time allotted to make a complete list of all instruments and systems that might be available for measuring properties pertinent to design of overlays. Hence, attention was focused on the major instruments available at the present time.

MEASUREMENT SYSTEMS

A pavement rehabilitation strategy requires the identification and selection of those pavements to be considered for attention as well as the type of action to be performed. The need to rehabilitate pavements arises from a variety of causes. First, unsatisfactory serviceability from the standpoint of riding qualities may be of prime consideration. Second, the need to correct structuaal deficiencies and the need to correct inadequate skid properties are other reasons for requiring pavement rehabilitation. Very often, several of these deficiencies might be present in a given situation.

Evaluation Devices

The type of device that can be adopted to fit a given situation is dependent on the reason for the pavement rehabilitation. For example, if riding quality is the prime factor under consideration and if a specific pavement must be overlayed, a deflection device might be sufficient for determining the required thickness of overlay. On the other hand, if severe distress to a given pavement section is noted, it might be desirable to make a detailed study by evaluating the physical properties of all of the pavement components. This becomes a diagnostic procedure to determine the cause of the pavement defect. Redesign of the pavement structure, then, is based on these data.

Table 4 shows a list of evaluation devices that were considered during the group's deliberations. The instruments are categorized in two ways. First, they are grouped on their basic principle of operation (deflection, curvature) and second whether the device is intended to be a diagnostic instrument or a rapid survey device.

DESCRIPTION OF DEVICES

Plate Bearing Test

In the plate bearing test a static load is applied to a circular plate and the displacement is measured. The size of plate used depends on several factors; for airports a standard 30-inch diameter plate is common and for highways plates 18 to 24 inches in diameter are often used. It is possible to determine the modulus of elasticity by this test.

Deflection

Benkelman Beam--This method uses a moving wheel load and pivoted beam. The deflection and the radius of curvature can both be determined by using the Benkelman beam. The device has such widespread use that it needs little description.

California Deflectometer--This device is essentially an automated Benkelman beam. Measurements are obtained while a loaded truck is moving. The device records maximum deflection. Because of mechanical operation downgrade measurements are not taken, and for safety purposes curves are not measured.

LaCroix Deflectometer--This system was developed in France and is used extensively in many countries. The transporting vehicle provides the loads, which approach a frame sitting on the pavement. The frame is equipped with lever arms attached to transducers that measure deflections as the wheel load approaches. The frame is then moved forward automatically and positioned to repeat the cycle. The system will measure and record deflections under both rear wheels at 12-foot intervals. The method is similar to the Benkelman beam but is different in beam length.

Curvature

An instrument developed in South Africa measures radius of bending under a wheel load. The device is simple to operate and portable. It is frequently used in conjunction with the Benkelman beam.

Impact

These methods employ a dropping weight to impart a shock wave to the pavement structure. This wave is measured and converted to deflection. Instruments employing impact are in use in Washington, France, Czechoslovakia, and Germany.

Washington state uses a small suitcase size device that employs a drop hammer similar to the one used in the proctor compaction test and that gives a deflection reading. A larger automated traveling device is being developed.

Vibratory

Shell--This is a device that uses a heavy or light vibrator to excite the pavement through an interchangeable size plate and load. Variable deflections can be obtained by using varying frequency. The instrument measures larger moduli with success.

Road Rater--This device is truck-mounted. A plate placed on the pavement is excited by a hydraulic system. Deflections at the plate and at intervals beyond the loaded area are obtained.

Dynaflect--The device is trailer-mounted and similar in principle to the road rater except dual steel wheels are used to load the pavement. Vibration is obtained with an eccentric cam.

Cox--A prototype model is being developed that is similar in operation to the road rater.

Wave Propagation

Several devices are in use to evaluate airport pavements. Wide frequencies are used to propagate waves into the pavements under relatively heavy loads. Devices are being used by the Waterways Experiment Station and the Air Force.

PROPERTIES MEASURED AND ADVANTAGES AND LIMITATIONS

Table 5 shows a summary of the systems previously discussed along with additional data regarding their adaptability for a variety of conditions. The data in Table 5 are self-explanatory inasmuch as the properties measured and how these data are utilized in design of overlays are outlined. One of the factors shown is whether the system is adaptable for rapid survey or whether it is a detailed diagnostic measurement system. Some of the systems outlined in the table are both, inasmuch as they can be used on a large-scale survey, but they have the added advantage of giving detailed information wherever desired.

Information given in Table 5 regarding speed of operation is the basis for a decision on whether the system is adaptable for rapid survey. On the other hand, it must be recognized that some of the systems (for example, the Shell vibrator) can be used for a variety of reasons; the information obtained from the device is dependent on the particular need in question at any instant of time. Likewise, the large-scale instrument developed by the Corps of Engineers is adaptable for rapid survey of airport pavements, and at the same time it can apply varying forces and can be used to determine basic properties of the pavement components.

Survey Equipment

It was agreed by those present that the LaCroix and the California deflectometers are well adapted for obtaining rapid survey data. The Dynaflect has high potential because it has been evaluated under a variety of conditions. The road rater shows considerable promise in this area as well. These instruments are immediately implementable and are available for purchase on the open market.

Future developments that might be investigated for improvement of rapid survey equipment include

1. Remote sensing,
2. Laser measurements of deflection and profile, and
3. Photographic techniques.

Laser Technology for Deflection Measurements--Application of new laser technology to measurement of surface deflection is currently under preliminary development jointly by three Seattle firms. It is expected that a basic unit can be attached to the underside of a loaded truck or aircraft to measure deflection caused by moving wheel loads. The procedure would be as follows:

1. The operating speed could be from 0 to 60 mph.
2. Deflection can be measured at any selected interval, say 50 feet.
3. The apparatus places a target at point of measurement, for example, on a spot in front of the vehicle.

Table 5. Summary of measurement systems.

CATEGORY	PRINCIPLE OF OPERATION	DEVICE	PROPERTY MEASURED	UTILIZATION		SPEED OF OPERATION OR NO. TESTS PER DAY	ADVANTAGES	DISADVANTAGES	DATA OUTPUT
				RAPID SURVEY	DIAGNOSTIC OR DESIGN				
A	Bearing Value	Plate	Modulus E		X	Very Slow	Measure E	Static Test	Immediate
B	Deflection	(a) Benkleman Beam	Deflection, Curvature	X	X	300 Per Day	Transportable, Realistic Loads		Direct Reading
		(b) California Deflectometer	Deflection	X	X	1800-2000 Per Day	Realistic Loads		Print Out
		(c) La Croix Deflectometer	Deflection	X	X	5000 Per Day	Realistic Loads		Digital
C	Curvature	So African	Curvature	X	X	300 Per Day	Small, Transportable		Immediate
D	Impact	Washington State	Deflection	X					
		Paris		X					
		German		X					
E	Vibratory	Shell	Deflection, Curvature, Modulus E		X	10 Minutes To 3 Hours Per Test	Multipurpose Large Force		Deflection Print Out
		Road Rater	Deflection, Curvature, Modulus E	X		600-800 Per Day	Varying Force	Small Force	Immediate
		Dynaflect	Deflection, Curvature, Dynamic E	X		400-600 Per Day			Immediate
		Cox (Under Development)		X					
F	Wave Propagation	Corps Engineers (Waterways Exp Station)	Deflection, Curvature, Dynamic E	X	X	300 Per Day	Large Force Multipurpose Varying Force		Automated
		Air Force (Under Development)	Deflection, Curvature, Dynamic E	X	X	300 Per Day	Large Force High Frequency		

4. The scanning laser locates the target.
5. Primary laser locks onto target and monitors vertical displacement as dual tires move over and past target. Several (5 to 10) measurements are made at each point in order to define maximum deflection and shape of basin.
6. Because laser measurements are made from a moving platform, the longitudinal profile is also monitored.
7. Data are recorded and analyzed or compiled on-board and are treated statistically as input to management system.

Photo Technology for Deflection Measurements--This is similar to the laser approach. New techniques in photogrammetry being developed at the University of Washington use 2 pairs of cameras carried by the loaded, moving vehicle. Analysis techniques permit accuracies to 0.001 inch deflection. Data, in the form of photographs, can be analyzed to varying degrees (vertical deflection at a point deflection basin in x and y directions or contoured, or continuous longitudinal) or an automated computerized stereoplotter at a central location.

Combined Field System--Once the deflection (i.e., structural capacity) can be included in field evaluation systems in sufficient detail (i.e., rapid data collection and analysis) along with other physical measurements, a single vehicle may be appropriate for covering many lane-miles in a short time.

Detailed Diagnostic Approach

The group deliberated at length on properties that should be measured in diagnostic studies. It was agreed that for future work it would be desirable for this type of instrument to measure the following:

1. Modulus of elasticity, E,
2. Rupture stresses and/or strains,
3. Thickness,
4. Fatigue properties,
5. Environmental factors, and
6. Pavement profile.

In addition to the above, the instrument must be relatively high speed and simple to operate.

The group recognized that a single instrument to measure all of the factors given would be extremely complex and that the development of this type of instrument may not be feasible in the near future. Nevertheless Group 2 recommends that emphasis be placed on development of this type of instrument and that, in particular, research money be funneled into development of instruments based on measurement of wave propagation and wave velocity.

SUMMARY

It became apparent that there are a wide variety of instrument and measurement systems available for evaluation of pavements. However, there is an urgent need to develop new systems that measure basic properties of the pavement components. The instruments currently available for rapid survey are serving adequately, but it would be desirable to develop a combined measurement system that uses laser and photographic techniques.

It is recommended that high priority should be given to development of high-speed accurate instruments for measurement of deflection (laser and/or photographic) as described above.

Of equal importance would be the development of diagnostic devices that measure basic properties of the pavement components. These properties include dynamic moduli and response of the pavement under dynamic loads.

MATERIALS AND TECHNIQUES

REPORT OF GROUP 3

R. G. Packard, Chairman
R. L. Hutchinson, Recorder

The group meeting opened with a general discussion of work covered by rehabilitation and no consensus evolved. Because of the obvious complexity of the problem it was agreed that the group would concentrate on pavement overlays and major repairs. The group also recognized that other groups were concerned with reflection cracking and skid resistance and thus did not concentrate on these problem areas but the group wishes to emphasize that these are major problem areas justifying a significant research in both improved materials and techniques. The group also recognized the desire to have a quantitative assignment regarding state of the art, research needs and implementation of each research effort. Because of the varying geographical needs, this was impractical and priorities were assigned only in a general way.

STATE OF THE ART

Throughout the group discussions it was evident that there has been significant research on materials and construction techniques, the results of which are applicable to the problem of pavement overlay and repair. It was also evident that the objective of many of these studies was specifically applicable to rehabilitation problems and thus the results are immediately implementable. However, many other studies are either general in nature or specifically for problems other than rehabilitation. The group recognized that the results of these latter studies need reanalysis for implementation in the rehabilitation problems. Thus some of the research needs identified here may be solvable from existing data. While these have not been specifically identified, they have generally been placed in the lower part of the overall priority needs. The group felt that the ability to achieve a rehabilitation program was fairly good by using conventional construction materials and techniques; however, from a cost-effectiveness standpoint the state of the art for a major pavement rehabilitation program was poor. They concluded that there has never been a well-designed and coordinated research effort specifically in the area of pavement rehabilitation.

PROBLEM AREAS AND CONSTRAINTS LEADING TO RESEARCH NEEDS

The following problem areas and constraints were identified, which lead to the need for research efforts in materials and techniques for pavement rehabilitation. They are listed in what is believed by the group the order of overall importance.

Rapid Construction and Repair

The need is to accomplish the overlay or repair with minimum interruption to normal traffic flow, especially in urban areas. Thus the importance of materials that can be applied with a minimum of effort and that are rapid setting so that the area can be quickly reopened to traffic cannot be overemphasized. Research needs leading to improvement of this problem area are generally considered to be of the highest priority.

Preparation and Strengthening of Existing Structures

The overall state of the art in the selection and use of materials and preparation of the existing structure prior to overlay was considered to be poor and an area where research could have significant returns in reducing the thickness and cost of overlays. Strengthening of subsurface layers and provision of uniform strength of the existing structure prior to overlay were stressed. Research efforts in this area were considered to be of second priority.

Materials and Techniques Resulting in Thinner Overlays

Overlay thickness is an important consideration at such points as overpasses, grade crossings, intersections, in urban areas where curb and guttering is used, and approaches to bridge decks, not to mention the cost and use of materials. The development of materials and techniques or the development of methods for optimal utilization of materials which will permit use of thinner overlays is important. Research studies in this area are considered to be a third order of priority.

Environmental and Energy Constraints

Minimizing the effects of the rehabilitation program in the environment and conservation of the energy required to produce materials are items that certainly warrant research. In fact, the group feels that constraints in these areas may well force the use of unconventional construction materials and may require use of construction techniques that minimize the use of energy. Research studies pertaining to this problem area are considered to be in a fourth order of priority.

Miscellaneous

Research studies considered important to the problem of pavement rehabilitation but not conveniently indentifiable in one of the above four problem areas or that are applicable to more than one of the problem areas are shown in this group of research needs.

The research needs identified by the group that are applicable to the problem areas are described below. To assist in the assessment of priorities, the research needs have been grouped as "most important" and "important" when applicable under each problem area. It is reiterated that the research problems were listed in the order of importance to the rehabilitation problem. Also, although mentioned earlier that the problem areas of skid resistance and reflection cracking were not addressed in detail, some research needs pertaining to these problem areas did arise and are included.

Skid Resistance and Reflection Cracking

1. Restoration of Skid Resistance. Less costly and more rapid techniques for texturing existing pavement surfaces than grooving are needed to restore skid resistance. Equipment currently under development that uses carefully spaced and controlled impact star drills should be evaluated.

2. Grooving of Plastic Concrete. Work has been accomplished in this area in the U.S. and England; however, the resulting grooves have been less than satisfactory. The process shows promise, and research is encouraged.

3. Development of Disposable Surfacing. Surfacings that could be removed and replaced easily and economically as a means of maintaining appropriate skid resistance is an area of applicable research.

4. Improved Asphalt Mix Design. It is believed that asphaltic concrete can be made more resistive to the problem of reflection cracking by the proper design of the mixture including the use of additives.

5. Membranes to Minimize Reflection. Limited success in controlling reflection cracking has been demonstrated by the use of petromat membranes on the existing surface prior to overlay. Further work with this and other membranes is needed.

6. Treatment of Cracks Due to Reflection. Until procedures are developed to eliminate reflection cracking, the problem of maintaining and treating these cracks is germane. Materials for sealing or other techniques for treating these cracks are needed.

Preparation and Strengthening of Existing Structures

1. Strengthening of Subsurface Layers. Significant savings could be achieved in required overlay thickness through the strengthening of subsurface layers in existing structures through such possible schemes as chemical injection or compaction through surface rolling. Some research has been conducted, but further work is needed in the development of techniques.

2. Improving Uniformity of Support of Existing Structures. Significant reductions in overlay thickness and improved performance of overlays may be achieved through the development of materials and techniques for providing uniform strength and surface grade to the existing structure prior to overlays. This would include improved leveling courses, improved mudjacking, or undersealing techniques or possibly the utilization of an economical underlay exhibiting uniform strength.

3. Improved Techniques for Determination of Uniformity of Existing Structure. Methods for rapid identification of voids, cracks, or other discontinuities that lead to nonuniform support are needed.

4. Widening Materials and Techniques. Research leading to the selection of compatible materials and construction techniques for widening existing pavements, either for edge rebuilding or for increasing width prior to overlay, are needed to eliminate the tendency for reflection cracking and differential vertical movement.

5. Bridge Deck Repair. Proper techniques for the removal of deteriorated material, preparation of disposed concrete surface or reinforcing steel and selection and application of replacement materials are needed as demonstrated by the variety of methods now in use that exhibit questionable performance.

6. Other indentifiable areas of research applicable to treatment of the existing structure are improved materials and methods for cleaning existing pavement surfaces, proper treatment or

restoration of surface and subsurface drainage, and treatment of asphaltic concrete exhibiting localized areas of deterioration from such things as fuel spillage, shoving, rutting, and excessive heat.

Materials and Techniques Resulting in Thinner Overlays

1. Improved Reinforcement for Concrete. Research has been conducted and some is underway in various methods for reinforcing and thus reducing the required thickness of concrete overlays. This includes use of fibers, continuous reinforcement (with and without elastic joints) and prestressing. Continuation of this research is encouraged as a means for minimizing the required thickness of concrete overlay.

2. Improved Bonding Procedures for Concrete Overlays. Bonding of concrete overlay to the existing concrete surface minimizes the required thickness of concrete overlay and permits reduction of the minimum thickness of concrete or reinforced concrete that can be used. Existing bonding procedures are slow, costly, and variable in effectiveness. Improved materials and techniques are needed.

3. Improved Asphaltic Concrete Mixtures. Thermal and reflection cracking are problems that often dictate the use of thicker than necessary overlays. Mix designs and/or additives that will better resist these environmental conditions but that will retain the stability, resist rutting, flow, shoving, and so on are needed to help minimize the required thickness.

4. Development of Fibrous Reinforced Asphaltic Concrete Mixes. Investigation of the use of randomly dispersed fibers in asphaltic concrete mixtures to improve the tensile and fatigue characteristics would permit thinner overlays. Some work has been done with asbestos fibers and steel fibers; however, other fibers such as polypropylene, nylon, and fiberglass should be studied.

5. Other identifiable research needs include improved load transfer for joints in concrete overlays and improved separation materials for non-bonded concrete overlays.

Environmental and Energy Constraints

1. Development of Mix Designs Using Asphalt Emulsions and Cement-Modified Asphalt Emulsions. Constraints in pollution and conservation of energy may prevent the use of asphalt which require heat and cut-backs which contain volatiles. Emulsions are possible replacements for asphalt to permit paving within the constraints and research into proper mix designs is needed.

2. Construction Techniques for Use of Emulsions and/or Cold Mixes. The use of asphalt emulsion mixes or cold mix asphalts present problems in handling, lay down and compaction, and techniques must be developed to provide a mass-produced pavement.

Miscellaneous Research Needs

1. Development of Expansive or Shrinkage-Compensation Cements. These type cements have distinct advantages for use with prestressed, fibrous or continuously-reinforced concrete overlays and for plain concrete where volume charge during curing results in excessive tensile stresses and cracks.

2. Development of Open Graded Asphaltic Concrete Mixes. Open graded mixtures have application for surface drainage to prevent hydroplaning, subsurface drainage layers as a deterrent to reflection cracking, etc. Research is needed to establish specification limits commensurate with useage such as required permeability, durability under chains and studded tires, and areas subject to freeze thaw or high temperatures. Definition of distress modes along with procedures for determination of thickness requirement are needed.

3. Paving Mastics. Improved mix designs, thickness design methods, and construction techniques are needed for asphalt paving mastics.

4. Gap Graded Asphaltic Concrete Mixes. These mixes show promise for increased flexibility and durability. However, mix designs along with design procedures for their use are needed.

5. Sulfur Concrete or Sulfur Additives. Some work in development of both sulfur concrete and use of sulfur as additives to asphalt has been accomplished. The results indicate possible use of sulfur as a cement or additive. However, additional research is required to delineate specific uses and resultant properties. Further research should recognize existing results indicating

that a sulfur-modified asphalt is susceptible to water, whereas it may be useful in asphalt paving mastics. It has also been found that sulfur is not compatible with certain aggregates when used as a cement under freeze-thaw conditions.

New Materials and Techniques

The following materials and techniques were identified by the group as applicable for research and useful for the overall rehabilitation problem. No effort was made by the group to assign priority or to assess probability of successes. Many of the items have been researched to the degree that development is highly probable but further research is needed before they become implementable in the rehabilitation program.

1. Impregnated Plywood Panels. Plywood panels impregnated with resinous materials can be rendered waterproof and durable and exhibit high strengths. Such panels have application to rapid repair tehhniques.

2. Polymers. Polymers used to improve the physical properties of conventional construction materials such as concrete and soil have promise for increasing the strength of both existing structures and new construction materials.

3. Elastomers. Elastomeric systems for use as waterproofing, stress relief layers, adhesives, joint sealers, and crack fillers have shown promise, and research should be continued with specific direction toward rehabilitation problems.

4. Lightweight, Precast, High-Strength Units. Modular units, similar to landing mat, that can be bonded to the surface for resurfacing and strengthening or used for localized repair have applications to the rehabilitation problem, especially rapid construction. Such units could follow the lead of airfield landing mats such as XM18, XM19, AM2, or Mo-Mat but should be configured for use on highway repair or resurfacing.

5. Resinous Impregnated Fiberglass. Such materials have been used for precast or cast-in-place units in airfields and could be configured for highway useage. Such units are extremely strong and cure rapidly.

5. Insulation Materials. Improved insulation materials that exhibit compatible properties and can be used between the existing structures and overlay material have application to minimize some of the environmental problems such as temperature gradients, frost penetration, and so on.

7. Utilization of Recycled Material. Research into use of recycled material needs to be continued since materials that exhibit superior strength, durability, or cost effectiveness may result, especially as the cost of disposing of material increases. Examples include use of shredded automobile bodies as concrete aggregate, shredded tires as asphaltic concrete aggregate, etc.

8. Utilization of Existing Structure Materials. Methods for the optimum utilization of the existing pavement materials need to be developed as a possible trade-off to overlays. Schemes such as the break up of existing concrete or asphaltic concrete and addition of new cements are examples. Another example would be the development of high-strength adhesives having the ability to penetrate cracks and joints of the existing structures.

STRATEGIES

REPORT OF GROUP 4

E. Barenberg, Chairman
R. Haas, Recorder

The charge to Group 4 was to consider the overall strategies for pavement rehabilitation including the economic consequences of the various alternatives for pavement rehabilitation and maintenance activities. The group discussed the components that should be included in evaluation of alternate strategies for pavement rehabilitation and the manner in which each of the components is likely to influence the consequences of decisions. In these discussions, consideration was given to identifying the state of the art, evaluating the current technology, implementability of the current technology, long-term and short-term research needs, and relevancy of strategies to pavement rehabilitation considerations.

DISCUSSION

The need for a concise definition of strategies became quickly apparent in the group deliberations. In the ensuing discussions, various members of the group indicated that strategies should have sufficient breadth to include procedures, types, timing, and scheduling of maintenance and rehabilitation activities; include economic evaluation of the scheduled activities; evaluate the cost effectiveness of the maintenance and rehabilitation activities, and be free of arbitrary constraints such as specific design life intervals. After considerable debate the following definition was agreed on:

> A strategy is a plan of action which embodies the continuing application of alternate solutions to pavement rehabilitation and maintenance needs.

The term continuing application of alternate solutions requires that complete, accurate, and current records be kept of the performance of the pavements in question. Such records should include data on pavement performance as well as complete and current records of all maintenance and rehabilitation activities applied to these pavements. The data collected should be monitored and evaluated on a continuing basis.

Although pavement monitoring and data collection and storage can be accomplished by manual means, the procedure should be set up to make data collection and evaluation as routine as possible. Use of computerized data storage and retrieval programs interfaced with automated data analysis programs is a good method of simplifying and automating such procedures.

Embodied within the definition of a strategy is an implied need for evaluation of the effectiveness of the proposed plans of action. This implies an ability to define user needs and to make benefit-cost analyses for the various strategies proposed. It was pointed out in the group deliberations, for example, that the term strategies means alternatives, the consequences of each alternative, and the proper information to make an assessment of each plan of action. It was noted by several of the group members that the cost and benefit components of the strategy analyses are not well defined. It was also emphasized that the user cost component for airfield pavements is significantly different from that for highway pavements. In particular, the delay costs for airfields during pavement rehabilitation can be very significant in the decision process. All of these statements point up the need for increased technology on the various components that make up a strategy and especially on such components of user needs, costs, and benefits to be derived.

There was considerable discussion by members of the group concerning the sensitivity of the alternatives to various components evaluated. Specifically, questions were raised with respect to such functional criteria as pavement roughness, safety, and serviceability level of the pavement and their influence on the criteria for evaluating the alternate plans. User costs, investment costs, and inflation were also discussed in terms of their effect on the decision criteria. Considerable discussion was held on what constitutes design life for pavement systems, what determines the effective design life, and what the significant differences are between design life and an analysis period for balancing cash flow. It was emphasized by several of the group that the design life or analysis period used in evaluating rehabilitation and maintenance alternatives can have a significant influence on the decision's reach and that efforts should be made to eliminate the arbitrary 20-year design life favored by many engineers. Most members of the group agreed that the effect of design life should be one of the variables evaluated in selecting a rehabilitation strategy.

In some preliminary discussions, there seemed to be an implied assumption that a strategy was more or less a schedule for overlaying the pavements. Several members of the group emphasized the need to broaden the discussion to include all rehabilitation and maintenance operations that might be applied. Accordingly, the following partial list of possible activities was developed: This list was intended for illustrative purposes to indicate the range of activities that should be included in such analyses and was not intended as a complete list of alternate activities. In the development of these alternatives, preventive as well as corrective activities were considered.

1. Overlays;
2. Seal coats (surface and rejuvenating);
3. Major maintenance (crack sealing, patching, etc.);
4. Subgrade and subbase strengthening through drainage;
5. Subgrade and subbase strengthening through nondestructive alteration such as intrusion injection, electro-osmosis, etc;
6. Reconstruction (i.e., removal and replacement);
7. Manipulation and reworking of existing pavements (i.e., reprocessing);
8. Applications to concrete surfaces to improve strength and durability (e.g., resins);

9. Application of erodable surface types;
10. Surface grooving; and
11. Various means of protection of components from the elements (i. e., encapsulation, thermal blankets, etc.).

STATE OF THE ART OF SYSTEMS PROGRAMS

The preceding discussion clearly demonstrates the complex nature of the problem of selecting the most favorable strategy and the almost infinite number of possibilities to choose from. Because of the large number of variables, optimum strategies can best be determined by use of programmed analyses and high-speed computers. Several such programs have been written and are being applied in the life cycle evaluation for pavement design and rehabilitation. Among the operational programs available are the following:

1. Texas Highway Department Program for Rigid Pavement Design, Research Report 123-5, Texas Highway Department, 1970.
2. Texas Highway Department Program for Flexible Pavement Design, Research Report 123-18, Texas Highway Department, 1973.
   Texas Highway Department Pavement Design System, "Part 1: Flexible Pavement Designers Manual," Highway Design Division, Texas Highway Departeent, 1972.
3. Flexible Pavement Design and Management-Systems Formulation (SAMP-6), NCHRP Report 139, 1973.
4. Synthesis for Rational Design of Flexible Pavements (VESYS II), Report of FHWA by J. E. Saussau, F. Moavenzadah, and H. K. Findakly, M.I.T., 1973.
5. Utah Highway Department Program for Determining pavement Maintenance and Rehabilitation Needs. Utah's System for Planning Road Improvements, Utah State Highway Department, 1970.
6. Ontario, Canada Program for Evaluating Pavement Design and Rehabilitation (Ref. Unknown).
7. U. S. Army CERL (LIFE-1) Program for Life Cycle Design of Airfield Pavements (Both Flexible and Rigid considered in the same program). "User Manual for LIFE-1 Computer Program." J. Willmer, P. McManus, E. Marvin, 1973.

Incorporated in the programs listed are several approaches to optimization of pavement rehabilitation and maintenance activities. Obviously some programs are more sophisticated than others, and the more sophisticated programs include components of optimization of strategies that the less sophisticated programs do not. Each of these programs is oriented toward specific facility, organization, or regional needs. Some of the programs relate more appropriately to network level analyses to be applied at high management levels.

STATE OF THE ART AND RESEARCH NEEDS

Development of alternate strategies for pavement rehabilitation implies an ability to project the performance trends with reasonable accuracy and to predict the consequences of the maintenance and rehabilitation activities. The technical aspects of projecting pavement performance and rehabilitation activities were covered in other group meetings and were not discussed in detail in the group meetings. Several of the factors that should be considered in determination of an optimum strategy are listed below.

1. Effect of rehabilitation activity: methodology and material;
2. Pavement structural condition;
3. History including traffic, time, environment, and maintenance activities;
4. Construction control of rehabilitation activities;
5. Pavement functional condition (serviceability);
6. Stochastic aspects; and
7. Safety.

The following should also be considered: State of the art (overall poor, some aspects better than others), effort required to implement, and urgency of research effort.

Given an ability to project performance and the effects of rehabilitation and maintenance activities, the second component for evaluating pavement strategies is that of optimization. Optimization, as it applies to pavement rehabilitation, includes optimization models, criteria for optimization, analysis period, evaluation of benefits, costs, monetary policy and available resources (constraints), and geometric considerations.

These components of optimization were further broken into elements, and the state of the art and research needs were evaluated. Because of the differences in the level of the state of the art and urgency of research needs for highway and airfield pavements, two separate evaluations were made when required.

	State of Art	Implementation Effort	Research Needs
Optimization models	5	4	3
Optimization criteria			
Economic			
Highway	2	5	5
Airfiedd	4	5	2
Safety (highways and airfields)	4	2	3
Comfort			
Highway	4	2	1
Airfield (also related to safety)	2	5	4
Sociopolitical (highways and airfields)	1	1	2
Environmental impact (highways and airfields)	3	2	3
Optimization time of analysis	4	4	1
Benefits (highways and airfields) including economic, social, safety, comfort, and environmental	3	5	3
Costs			
Agency (city, state, county)	5	1	1
User			
Highway	2	4	5
Airfield	4	2	1
Monetary policy (discount rate, inflation)	5	1	1
Resource availability (money, materials, manpower, equipment)	5	2	2

SUMMARY

In projecting performance trends and consequences of pavement rehabilitation and maintenance activities, considerable attention must be given to their stochastic nature in terms of serviceability with time and in terms of serviceability variations along the length of a pavement segment. In many instances it may be more practical to rehabilitate short pavement segments rather than attempt to upgrade an entire pavement section.

The need for management information or feedback systems to continuously update the pavement functional condition was again emphasized.

Development of an optimum strategy is a necessary but not a sufficient condition for its successful application. The proposed strategy and its projected consequences must also be communicated to the operational personnel and especially to maintenance personnel.

Strategy analysis should be applied to network analysis of pavement systems, individual pavement section, and alternate transportation modes. Current transportation policy should be taken into account when pavement strategies are evaluated.

REFLECTION CRACKING

REPORT OF GROUP 5

D. R. Schwartz, Chairman
C. S. Hughes, Recorder

RELEVANCY OF REFLECTION CRACKING TO PAVEMENT REHABILITATION

Problems associated with cracks in existing pavement reflecting through new surfacings are important considerations in pavement rehabilitation and can play an important part in the degree of success obtained in the rehabilitation process. This is particularly true in the case of rehabilitation with bituminous concrete overlays, which has been and continues to be the predominant form of rehabilitating highway pavements. Reflection cracks can be the source of early deterioration in bituminous concrete overlays, causing accelerated maintenance and reducing the useful service life of the rehabilitated pavement. Deterioration in the form of raveling and spalling of the surfacing occurs at reflection cracks in bituminous concrete resurfacing of flexible pavements. In addition to these, closely spaced parallel cracks and tenting, or humps, often occur at reflection cracks in bituminous concrete resurfacings over existing rigid pavements. Another concern is intrusion of water into the crack opening, which can cause loss of bond between the surfacing and pavement and can result in pumping and a decrease in subgrade support. Many remedies have been proposed and tried over the years, but as yet there is no proven method that can be used to guard against reflection cracking in bituminous concrete overlays.

Reflection cracking in portland cement concrete (PCC) overlays also is an important consideration in this method of rehabilitation, which has been used some in the highway field and to a much greater extent in airport work. The research work in this area, however, has been more fruitful, and methods have been developed and are being used to successfully control reflection cracking in PCC overlays of rigid pavements.

STATE OF THE ART

Many techniques and materials have been tried over the years to reduce or eliminate reflection cracking in bituminous concrete overlays. Some have worked well on particular projects, but none has completely eliminated the cracking and none has been consistently successful in repeated tests. Lack of definition of the variables involved in the overlay designs and construction and in the pavements being overlaid has hindered the beneficial use of the results in predicting why a method worked well in one area but not in another or worked well on a particular project in an area and not on another project in the same general area.

As indicated in the state-of-the-art paper by Sherman, considerable work has been done on reflection cracking, most of which has been empirical and concerned with field tests of proposed methods or treatments. None of the work has provided a proven method whereby a bituminous concrete overlay can be designed with full assurance that reflection cracking will not develop.

The research efforts to control reflection cracking when existing concrete pavements are overlayed with PCC have provided three satisfactory solutions that have been implemented. In one solution, the same joint spacing is used in the PCC overlay as in the existing pavement, with the new joints formed directly over the joints in the existing slab.

In the second solution, where a different joint pattern is used, a 1- to 4-inch thick stress-relieving course with a stiffness much less than that of the pavement or overlay is placed between the pavement and overlay to absorb the volume change forces and prevent their transfer into the overlay.

The third solution is to place a continuously reinforced concrete overlay over the existing pavement. Sufficient additional longitudinal reinforcement should be placed in the overlay to carry the volume change forces developed by the existing slab, or thin asphalt leveling course can be placed over the existing pavement to level the existing pavement and to serve as a bond breaker prior to construction of the overlay.

THRUST OF CURRENT RESEARCH

Research work being carried out at Ohio State University is using a mechanistic approach in studying reflection cracking in bituminous surfacings. The study has defined the stress and failure modes causing reflection cracking and has developed a mechanistic model that is now undergoing field verification. Successful completion of this work could provide a model fur use in design to eliminate the problems with reflection cracking in bituminous concrete overlays.

In addition, the NEEP Project 10 is concerned with reducing reflection cracking in bituminous overlays. This study involves 12 state highway agencies and includes a series of field trials of several different materials and techniques. Among those being tested are various forms of rubber and rubber additives, fabrics, special mixes, and granular layers.

RESEARCH NEEDS

The following needs for research to eliminate or minimize reflection cracking in bituminous concrete overlays are recommended in order of descending priority:

1. Past research has been concerned primarily with field trials to test various treatments proposed to control reflection cracking without any effort to define the factors causing the cracking. Current research being conducted at Ohio State University on reflection cracking is taking a mechanistic approach to the problem. A more concerted effort in this direction is needed. A total solution to the problem can be obtained only after a thorough understanding of the mechanisms involved is reached.
2. There is a need for the establisnment of criteria and guidelines for use in data collection and analyses in connection with ongoing field tests of new remedies proposed for eliminating reflection cracking as part of the National Experimental Evaluation Program to provide more valid results and direct comparisons among the different tests. The guidelines should provide specific direction and standardization of data collection so that the information obtained will be useful for field verification of mechanistic models developed from research outlined above.
3. Preliminary results of current research indicate that reflection cracking develops from failure of the surfacing in tension due to temperature changes in the base and in shear due to loads imposed by traffic. This suggests a need for research to improve the strength characteristics of bituminous concrete mixtures. Mixtures with improved tensile, shear, and elastic properties at low temperatures could eliminate or greatly reduce problems with reflection cracking.
4. The above 3 research needs are directed toward obtaining information for use in eliminating reflection cracking. Whereas this is the most desirable approach, there undoubtedly will be many instances in which it may not be economically feasible to eliminate the cracking. Reflection cracking in itself does not appear to reduce the level of service being provided by an overlaid pavement. Reduced service results from the subsequent deterioration of the bituminous overlay that occurs at reflection cracks. This suggests a strong need to define the mechanisms involved in the development and progression of deterioration that occurs at reflection cracks in bituminous concrete surfaces and to develop procedures for corrective measures.

OTHER SPECIAL PROBLEMS

REPORT OF GROUP 6

J. O. Kyser, Chairman
D. C. Mahone, Recorder

Four specific areas of concern, applying to both highways and airfields, were identified to be within the charge of the group, and their evaluation and recommendation are presented as follows.

REHABILITATION OF THERMALLY CRACKED PAVEMENTS

Thermal cracking of asphalt pavements is recognized in Canada and the northern United States. Engineers in areas where thermal cracking develops need to specify special asphalts to reduce or eliminate thermal cracking, especially when working with overlays. Care should be taken to use asphalts that would not cause other problems. Canada has experienced success in specifying viscosity (140 F) and penetration (77 F) in combination. Thermal cracking of portland cement concrete pavements is known.

State of the art for new construction is available for Canadian conditions; its applicability to U.S. conditions rates as 3 on a scale of 1 to 5. There is no implementable procedure for the rehabilitation of thermally cracked pavements that may occur in the United States.

Research needs are as follows:

1. Establish the extent of the problem in the United States, try available techniques

for new construction, and develop new techniques for implementation to reduce thermal cracking of overlays.
2. Where thermal cracking occurs, there is then great urgency to develop techniques and procedures for rehabilitation.

Rating for research needs is 5 (highest).

REHABILITATION OF HIGH-VOLUME AND HIGH-SPEED PAVEMENTS

The rehabilitation of high-volume and high-speed pavements poses special problems. These are related to user costs, construction delays, strengthening of an adjacent lane without loss of crown, and safety of operations. There is some information on techniques that can be used, but a great effort is required in research needs:

1. Handling high-volume, high-speed traffic during rehabilitation,
2. Developing more rapid repair methods for all types of material, and
3. Developing an overall economic strategy relative to handling traffic and rapid repairs.

Rating for research needs is 5.

MOISTURE DRAINAGE

Moisture drainage had been presented as a special problem to Group 6 for consideration in the rehabilitation research effort. Our consensus is that much knowledge on the contribution and effects of moisture to rehabilitation is available but that its implementation has been lacking. Therefore, research needs on this have the lowest priority.

ENVIRONMENTAL AND ECOLOGICAL RESTRAINTS

The rehabilitation of pavements in view of environmental and ecological restraints may pose some problems. These types of problems are related to the depletion of good aggregates, forcing us to use marginal material or even the material of an existing road. Shortages in petroleum distillates may restrict the use of cutbacks, as may some warrants based on air pollution. Air quality consideration may restrict use of certain equipment such as heater planers.

With the energy crisis and the large amount of fuel consumed in hot-mix materials, research should be done on decreasing the amount of fuel consumption and/or usage of different types of heating fuels, as well as research in the use of cold mixes that would be as serviceable and workable as hot mixes, possibly by using chemical compositions to assist the cold-mix operation.

It is evident that another research need is development of alternative used of material resources in view of diminishing availability and restrictions placed on materials used and construction techniques.

Our rating of this need is of medium priority, 3.

WORKSHOP FORMAT

During the last 2 decades the total mileage of heavy duty highway systems placed in service has greatly increased. Additionally, there continues to be a general increase in traffic loadings on all highway systems. A corresponding trend exists in construction and use of airfield runway systems. For reasons of structural adequacy, riding quality and safety, maintenance of pavement systems has taken on great significance. Limited funds available for pavement maintenance emphasize the need for a careful evaluation of available methodology and needed research to ensure the optimum use of these resources.

In general there are four considerations in determining the need for pavement maintenance:

1. Structural deterioration leading to a reduction in load-carrying capability,
2. Unsatisfactory riding quality,
3. Inadequate skid resistance, and
4. Surface deterioration such as raveling.

All four needs are important, and frequently two or more occur simultaneously. However, the area that requires considerable study at present is concerned with structural adequacy and the methodology necessary for structural evaluation of pavement systems and overlay thickness design.

SCOPE

The workshop shall address the broad field of pavement rehabilitation and related strategies applicable to both highway and airfield pavements. Specific areas of concern shall be explored in detail in individual sessions as indicated in the program outline.

PURPOSE

The workshop will bring together a group of the most qualified engineers and scientists available to accomplish the following objectives:

1. Assemble and present pertinent information concerning the rehabilitation of highway and airfield pavements,
2. Appraise and evaluate pertinent information,
3. Identify implementable information that can readily be incorporated into practice, and
4. Identify areas needing further research and formulate a research framework to guide the Federal Highway Administration in the development of a research program in structural rehabilitation of pavement systems.

METHOD OF ACCOMPLISHMENT

To best accomplish the objectives outlined, it is considered desirable to hold the workshop for 3 1/2 days of formal working sessions in which individuals would meet and exchange information within the format described. The benefits to be derived from a general review of ongoing research make it desirable to hold the workshop subsequent to the FHWA-sponsored Federally Coordinated Program Research Progress Review, scheduled to be held in San Francisco, California, September 17-18, 1973.

A number of authors will be invited to prepare state-of-the-art reports for early distribution to the attendees. The total attendance will be limited to 50 engineers and researchers, plus representatives from FHWA and other government agencies. To ensure that all areas of concern are addressed, the project advisory committee, in consultation with the FHWA project manager, will ensure that a balance of disciplines are invited to participate.

It is anticipated that the output of the workshop will consist of four parts: state-of-the-art reports presented in formal sessions; summaries of session deliberations by session chairmen and recorders; appraisal and evaluation of workshop reports and proceedings; and research needs framework.

The general format suggested for the meeting is illustrated in Table 1. The proposed format for each of the sessions and the state-of-the-art reports is shown in the accompanying program outline. State-of-the-art reports, prepared in advance and sent to meeting participants by September 1, 1973, will be presented in Sessions I, II, and IV.

TABLE I GENERAL FORMAT FOR MEETING

Period	Wednesday Sept. 19, 1973 1	Thursday Sept. 20, 1973 2	Friday Sept. 21, 1973 3	Saturday Sept. 22, 1973 4
Morning	Session I (State of Art)	Session IV (State of Art)	Session VII (Group Meetings A-E)	Session IX (Summary)
Afternoon	Session II (State of Art)	Session V (Group Meeting I-VI)	Session VIII (Group Meeting I-VI	Advisory Committee Meeting
Evening	Session III (Groups Meetings A-E)	Session VI (Reports & Discussion)	Advisory Committee and Working Period in Preparation for Session IX	

PROGRAM

Wednesday, September 19, 1973
SESSION I (8:30 a.m. to 12 noon)

Opening Session

This session will provide a definition of the problem and establish a general overall framework for subsequent deliberations.

C. L. Monismith, Chairman	Time (minutes)
1. Chairman's Introductory Remarks	15
2. A General Framework for Pavement Rehabilitation W. R. Hudson and F. N. Finn	50
3. Discussion	25
4. Break	20
5. Surface Evaluation of Highway and Airfield Pavements R. C. G. Haas	50
6. Discussion	40
7. Workshop Procedures	10
Total	210

Wednesday, September 19, 1973
SESSION II (1:30 to 5:00 p.m.)

Structural Evaluation of Pavements and Overlay Design

J. H. Hewett, Chairman	Time (minutes)
1. Structural Evaluation of Highway Pavements and Overlay Design R. A. McComb	50
2. Discussion	45
3. Break	20
4. Structural Evaluation of Airfield Pavements and Overlay Design M. W. Witczak	50
5. Discussion	45
Total	210

BACKGROUND TO SESSIONS III, V, VII & VII

Small groups of attendees, representative of various disciplines, will direct their efforts in these sessions toward the formulation of a research program that will produce solutions to the problem of structural rehabilitation of pavements. The overall problem has been subdivided into 11 topics, one of which has been assigned to each small group, and the group is expected to devote its attention to the topic to which it has been assigned during the time allotted. Each topic will be addressed twice.

A chairman and reporter will be appointed for each group. The chairman will be expected to have an outline of discussion points for his group, prepared from his own experience and his review of the manuscripts distributed prior to the meeting. The reporters for each small group will prepare a summary for consideration in subsequent sessions.

The format proposed for these sessions may be viewed as a structuring of the problem into two categories for small group discussions as follows:

Category 1. Rehabilitation Techniques, Strategies, and Problems

Group	Topic
1	Condition Surveys
2	Measurement Systems
3	Materials and Techniques
4	Strategies
5	Reflection Cracking
6	Other Special Problems

Category 2. Thickness Selection Procedures

Group	Topic
A	Modification of Structural Design Procedures
B	Layered Elastic Systems Procedures
C	Deflection, Curvature, and Stiffness Based Procedures
D	Other Procedures
E	Ideal Procedures

Individuals assigned to participate in Groups A through E will be reconstituted so that one or more individuals from these groups will be assigned to participate in each of Groups 1 through 6.

Wednesday, September 19, 1973
SESSION III (7:00 to 10:00 p.m.)
and
Friday, September 21, 1973
SESSION VII (8:30 a.m. to 12 noon)

Thickness Selection Procedures

Group	Topic	Chairman	Recorder
A	Modification of Structural Design Procedure	W. J. Kenis	J. L. Brown
B	Layered Elastic Systems Procedures	M. W. Witczak	L. Santucci
C	Deflection, Curvature, and Stiffness Based Procedures	W. A. Moore	D. E. Peterson
D	Other Procedures	R. LeClerc	M. Womack
E	Ideal Procedures	D. I. Hanson	M. Herrin

NOTE

The discussions in each group shall include the following subjects:

1. Required input,
2. Assumptions associated with processing,
3. Processing of data,
4. Output,
5. Advantages and disadvantages, and
6. How remaining life of existing section is estimated.

Thursday, September 20, 1973
SESSION IV (8:30 a.m. to 12 noon)

Special Problems

This session will attempt to delineate special problems that impinge on pavement rehabilitation decision processes and maintenance strategies. Brief state-of-the-art papers will be presented on selected special problems followed by a general discussion open to all participants.

D. I. Hanson, Chairman		Time (minutes)
1.	Introduction	10
2.	Reflection Cracking, George B. Sherman	15
3.	Thermal Cracking, W. Phang and K. O. Anderson	15
4.	Moisture and Drainage, G. W. Ring	15
5.	Discussion	45
6.	Break	20
7.	Urban Area Problems, L. G. Byrd	15
8.	Maintenance Problems	
	Highways, T. W. Smith	15
	Airfield, D. I. Hanson	15
9.	Discussion	45
	Total	210

Thursday, September 20, 1973
SESSION V (1:30 p.m. to 5:00 p.m.)
and
Friday, September 21, 1973
SESSION VIII (1:30 p.m. to 5:00 p.m.)

Rehabilitation Techniques, Strategies, and Problems

Group	Topic	Chairman	Recorder
1	Condition Surveys	K. H. Dunn	D. Lambiotte
2	Measurement Systems	E. J. Yoder	W. L. Gramling
3	Materials and Techniques	R. G. Packard	R. L. Hutchinson
4	Strategies	E. J. Barenberg	R. C. G. Haas
5	Reflection Cracking	D. R. Schwartz	C. S. Hughes
6	Other Special Problems	J. O. Kyser	D. C. Mahone

Thursday, September 20, 1973
SESSION VI (7:00 p.m. to 10:00 p.m.)

General Session

C. L. Monismith, Chairman
J. A. Deacon, General Reporter

It is anticipated that the general format for this session will be presentations of 15-minute duration by each group chairman followed by a discussion of his report. A general reporter will then summarize the material presented. This presentation will be followed by a general discussion open to all participants.

Saturday, September 22, 1973
SESSION IX (8:30 a.m. to 12 noon)

Concluding Session

C. L. Monismith, Chairman

1. Reports by Small Groups
2. Break
3. Advisory Panel Report
4. Discussion

5. Concluding Remarks

Saturday, September 22, 1973
(1:00 p.m. to 5:00 p.m.)

Post-Workshop Meeting of Advisory Committee

The advisory committee will meet the fourth afternoon to review and technically edit the reporters' statements and discussions and to prepare guidelines for authors of summary reports on the workshop proceedings and proposed research framework. A format for the preparation and review of the final report will be established.

ASSIGNMENT TO GROUPS

CATEGORY 1 - REHABILITATION TECHNIQUES, STRATEGIES AND PROBLEMS

Condition Surveys I	Measurement Systems II	Materials & Tech. III	Strategies IV	Reflection Cracking V	Other Special Problems VI
K. Dunn, Chm.	E. Yoder, Chm	R. Packard, Chm.	E. Barenberg, Chm.	D. Schwartz, Chm.	J. Kyser, Chm.
D. Lambiotte, Rec.	W. Gramling, Rec.	R. Hutchinson, Rec.	R. Haas, Rec.	C. Hughes, Rec.	D. Mahone, Rec
R. LeClerc	W. Moore	M. Herrin	W. Kenis	D. Hanson	J. Brown
W. Hudson	J. Green	M. Witczak	D. Peterson	G. Sherman	K. Anderson
R. Jump	S. Lanford	T. Smith	W. Phang	J. Labra	J. Shook
P. Velz	R. Terrel	J. Deacon	P. Melville	K. Majidzadeh	F. Young
R. Olsen	F. LaBelle	J. Hewett	R. Barksdale	J. Bryden	M. Sharma
G. Tessier	R. Berry	J. Rice	T. Pasko	W. Head	R. Jimenez
F. Moavenzadeh	B. Rao	L. Oehler		J. Lai	G. Ring
M. Womack	G. Argue	L. Santucci			J. Hode Keyser
L. Price	J. Matthews	J. Burke			

CATEGORY 2 - THICKNESS SELECTION PROCEDURES

Mod. Struct. Des. Procedures A	Layered Elastic Sys. B	Deflec., Curvature Stiffnes Based Pro. C	Other Procedures D	Ideal Procedures E
W. Kenis, Chm.	M. Witczak	W. Moore, Chm.	R. LeClerc, Chm.	D. Hanson, Chm.
J. Brown, Rec.	L. Santucci, Rec.	D. Peterson, Rec.	M. Womack, Rec.	M. Herrin, Rec.
D. Lambiotte	E. Barenberg	W. Gramling	K. Dunn	R. Hutchinson
R. Packard	D. Mahone	C. Hughes	E. Yoder	R. Haas
W. Hudson	K. Anderson	J. Kyser	D. Schwartz	G. Ring
T. Smith	S. Lanford	G. Sherman	W. Phang	J. Rice
J. Deacon	R. Terrel	R. Jump	L. Price	J. Labra
G. Argue	B. Rao	F. LaBelle	J. Matthews	K. Majidzadeh
G. Tessier	J. Shook	R. Berry	P. Velz	F. Moavenzadeh
L. Oehler	J. Lai	J. Hewett	R. Olsen	M. Sharma
W. Head	J. Green	F. Young	J. Bryden	J. Hode Keyser
R. Jimenez		P. Melville	J. Burke	
T. Pasko		R. Barksdale		

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