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*Pavement Design, Management, and Performance*

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PART 1

## 1992 TRB Distinguished Lecture

CARL L. MONISMITH

PART 2

## Developments in Flexible Pavement Design



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# Transportation Research Record 1354

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# PART 1

# 1992 TRB Distinguished Lecture

# Part 1
# Foreword

In 1990 the Transportation Research Board Executive Committee approved the establishment of the Distinguished Lectureship Series to recognize the career contributions and achievements of an individual in one of four areas covered by the Board's Technical Activities Division: transportation systems planning and administration (Group 1); design and construction of transportation facilities (Group 2); operation, safety, and maintenance of transportation facilities (Group 3); and legal resources (Group 4).

Those selected are provided a forum at the TRB Annual Meeting to present an overview of their technical area, including evolution, present status, and prognosis. Carl L. Monismith, Robert Horonjeff Professor of Civil Engineering at the University of California, Berkeley, is the first to be honored with the TRB Distinguished Lectureship. His lecture, entitled Analytically Based Asphalt Pavement Design and Rehabilitation: Theory to Practice, 1962–1992, was sponsored by Group 2 and presented at the 1992 Annual Meeting. It is published in this Record.

# Carl Monismith, 1992 TRB Distinguished Lecturer

CARL L. MONISMITH

Carl L. Monismith earned bachelor's and master's degrees in civil engineering from the University of California, Berkeley. He has been a member of the staff of the Department of Civil Engineering and the Institute of Transportation Studies throughout his career, serving as Chairman of the Department of Civil Engineering from 1974 to 1979. He is the first to hold the Robert Horonjeff endowed chair in the Department.

Monismith is internationally recognized for his work in pavement design and rehabilitation and asphalt paving technology. He has published extensively, his papers having received awards from the Association of Asphalt Paving Technologists (AAPT), the Transportation Research Board, and the American Society of Civil Engineers (ASCE). He was elected to the National Academy of Engineering in 1980. In 1988 he received the James Laurie Prize from ASCE for contributions to transportation engineering, and in 1989 he was elected to an Honorary Membership in AAPT.

He serves or has served as a consultant on pavement research and design to the Asphalt Institute; Chevron Research Company; U.S. Army Corps of Engineers Waterways Experiment Station in Vicksburg, Mississippi; Transport Canada-Air; Woodward Clyde Consultants; Bechtel Corporation; ARE, Inc.; and the U.S. Air Force.

Active in many professional societies, Monismith has served as President of AAPT, President of the San Francisco Section of ASCE, and Chairman of the Pavement Design Section of the Transportation Research Board. He has been active for 30 years in TRB, in which he currently serves as a University Representative and a member of the Committee on Flexible Pavement Design. He is a registered civil engineer in California. Hallmarks that distinguish Monismith during his career in transportation include the following:

- His goals have been to raise educational standards, improve the engineering profession through research and its implementation, develop leaders, and help other engineers cope with new situations. Further, he has accomplished these goals in a manner that serves as a model for all engineers.
- He and his associates have helped lead the asphalt paving industry into a new era. Pavement failure models were studied—fatigue, thermal cracking, rutting, shear. Mechanistic models were developed to optimize design. Layered elastic theory and finite-element analysis were used in modeling and to solve specific problems. Resilient modulus, fatigue, and creep tests were developed to better characterize materials. Maintenance and rehabilitation techniques were classified and their effectiveness and service life were determined. The new technology permitted engineers to determine what maintenance technique was most appropriate to extend pavement service life and when to do the work. Pavement management systems were developed and are now used extensively to study the impact of changes—in financing, in vehicle loads, in materials, and in timing for maintenance and rehabilitation.
- An outstanding teacher, he shares his ideas and stimulates students to achieve. Many have become leaders in education, engineering, and research.
- He has authored or coauthored over 160 technical papers, winning many awards.
- He participates in the work of many organizations and technical societies—to learn, share, test new concepts, implement research, and serve.

Professor Monismith is a rare breed—inspiring educator, outstanding researcher, prolific writer, effective administrator, and leader in many technical matters.

# Analytically Based Asphalt Pavement Design and Rehabilitation: Theory to Practice, 1962–1992

CARL L. MONISMITH

## INTRODUCTION

The subject of this lecture is the development of analytically based asphalt pavement design and rehabilitation during the 30-year period from 1962 to date, development that received impetus from the First International Conference on the Structural Design of Asphalt Pavements, held at the University of Michigan in 1962.

Why this subject, other than the fact that this has been my area of engineering research, at least for this 30-year period? When one considers the amount of money expended for pavements in general and asphalt mixes in particular, the importance of research in this general area becomes apparent. Consider, for example, a 1-mi stretch of freeway consisting of three lanes in each direction with shoulders; the cost for an 8-in.-thick layer of asphalt concrete alone would be of the order of $3 million. Considering then the existing street and highway network as well as airfield pavements, parking facilities, and so on, it is possible to understand why expenditures for asphalt mixes are of the order of $10 billion to $12 billion per year. If research could save 1 percent of these costs, the 5-year, $50 million asphalt research program on which the Strategic Highway Research Program (SHRP) is now embarked would be paid for in approximately 4 months.

Add to the asphalt mix costs those for other pavement components as well as the potential savings in user costs resulting from improved pavement performance, and one develops an appreciation for the necessity of continued research on pavements.

The perspective presented herein has been influenced strongly by research at the University of California at Berkeley during this period, research performed by many who have contributed significantly to the field. Moreover, the interrelation of asphalt pavement design and rehabilitation and asphalt-aggregate mix design is emphasized; they must be considered together to develop optimally performing pavement structures, whether new or rehabilitated.

Although 1962 has been selected as the starting point for analytically based design as we know it, four developments before this time had a significant influence: the early analytical work of Burmister (*1*), the development of a deflection-measuring device (the Benkelman beam) during the WASHO Road Test (*2*), the research by Hveem on pavement deflections and fatigue failures (*3*), and the AASHO Road Test (*4*).

A brief summary is given of available analysis procedures that have been developed during this period, analysis procedures made possible by the advent of the electronic computer. Included are references to available multilayer elastic and viscoelastic analyses using closed-form solutions and finite-element idealizations. Although the majority of these solutions are based on so-called static load response, a recently developed computer solution permits treating the load and pavement response dynamically.

A simplified framework for pavement analysis and design is shown in Figure 1. Some aspects of this process will be discussed, as outlined in the following paragraphs.

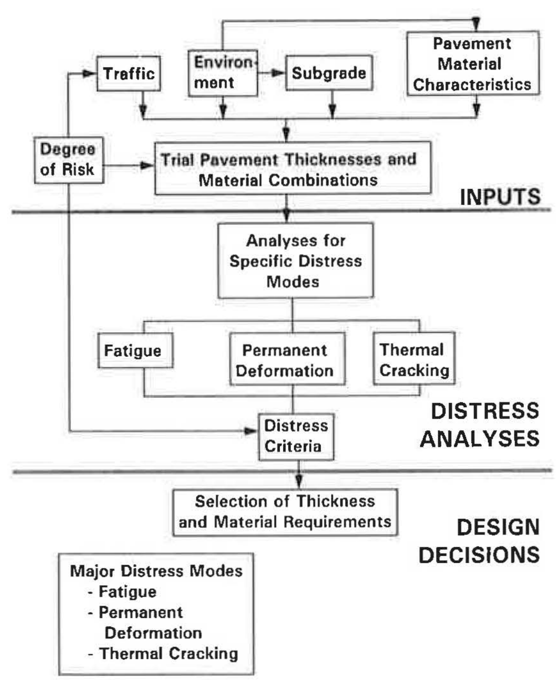


**FIGURE 1 Simplified framework for pavement analysis and design.**

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Materials characterization, so necessary to supply the requisite input information for analytically based design and rehabilitation, is reviewed. A summary of constitutive relationships for stiffness (modulus) characteristics that can be used in the various analysis procedures is presented for fine-grained (subgrade) soils, untreated granular materials, asphalt- or modified-binder-bound materials and portland-cement-stabilized materials. Included are the effects of environment and stress on material characteristics.

The necessity for proper specimen preparation is emphasized, and some recommendations are included to ensure that the stress-strain characteristics of laboratory-prepared specimens will be representative of the materials as they exist in the pavement structure.

*Fatigue, permanent deformation*, and *thermal cracking* are the three major modes of distress that lead to a reduction in the serviceability of asphalt concrete pavements. So that the potential for these forms of distress can be analyzed within the framework of Figure 1, prediction methodologies as well as distress criteria are required. Methodologies associated with each of the three distress modes are discussed.

A number of design procedures have been developed that incorporate aspects of the various analyses and materials evaluation procedures described in this paper. Some of these are briefly summarized to illustrate the efficacy of this approach. In effect, the intent of the analysis and design process is to simulate, in advance, the expected performance of the asphalt pavement so that the optimum thicknesses of the various components can be selected and the available materials can be used effectively. Thus, it is possible for an engineer to use the information of the type presented and, interacting with a computer work station, to carry out for either new or rehabilitated pavements designs that range from relatively simple to complex, depending on the significance (and cost) of the particular project. Moreover, through this process, general guidelines and catalogues of information can be developed for future reference. Updating and modifications are possible as further research provides additional information. It must be recognized that pavement design and rehabilitation processes are not static, but must be amenable to improvements as research-based developments provide better ways to do the job.

In a discussion of this type, in which past is prologue, I have also included recommendations for what might be done in the next few years to expand our capabilities for pavement design and rehabilitation. One person can never provide an all-encompassing view; rather, suggestions like those included here together with recommendations from others provide a framework by means of which substantial advances can be made. It is my intent that the material included here be presented in that context.

## BACKGROUND AND FOUNDATIONS

Although there was considerable development in the pavement field before 1962, four studies from this period, in my opinion, have affected the development of analytically based design methodology.

A key development was the work of Burmister in the early 1940s, that is, his solutions for the response of two- and three-layer elastic systems to representative loading conditions (*1*). Although these solutions were limited to conditions at layer interfaces and the results were generally presented in graphical form, they nevertheless introduced the engineering community to the important concept of treating the pavement as a layered system. Comprehensive use of these solutions would have to wait approximately 15 years for the advent of the electronic computer.

A second important development occurred during the WASHO Road Test when Benkelman introduced the Benkelman beam, which permitted pavement deflections to be measured under slow-moving wheel loads (*2*). The Benkelman beam facilitated rapid measurement of pavement response, thus providing an early indication of future performance and a comparative measure against which to check calculated pavement response. It also provided an important tool for improved design of overlay pavements.

A third important development occurred in California. Using a General Electric travel gauge, Hveem had been investigating pavement deflections for a number of years before the WASHO Road Test. Publication of his research in 1955 (*3*), which is one of the most important papers in the pavement field, provided a strong link between pavement deflections and fatigue failures in the asphalt-bound portion of pavement sections. Hveem's work had, in my judgment, a most significant impact on the development of procedures to predict fatigue cracking using analytically based methodologies.

The AASHO Road Test, completed in 1961, is the fourth important development (*4*). Funded in the 1956 Interstate Highway Act, its cost of $29 million corresponds to the current cost of the SHRP program. It sparked a renewed interest in improved pavement design and provided the impetus for the development of many of the current analytically based design procedures. Under the excellent leadership of W. N. Carey, Jr., the AASHO Road Test provided another important contribution to the engineering community, since well-documented performance data were assembled and stored, permitting future researchers to have access to them. Performance predictions by the new analytically based procedures could be compared with actual field performance; reasonable comparisons confirmed the "engineering reasonableness" of the methodologies.

## PAVEMENT ANALYSIS

The use of multilayered analysis to represent pavement response, although developed by Burmister in the 1940s (*1*), did not receive widespread attention until the First International Conference on the Structural Design of Asphalt Pavements in 1962. Although some agencies utilized solutions for two- and three-layered elastic solids in their design methodologies [e.g., the U.S. Navy (*5*)], the use of these solutions was both limited and cumbersome.

At the 1962 conference, however, important contributions were made by Whiffin and Lister (*6*), Skok and Finn (*7*), Peattie (*8*), and Dormon (*9*). Both Whiffin and Lister and Skok and Finn illustrated how layered-elastic analysis could be used to analyze pavement distress. Peattie and Dormon presented a number of concepts based on such analyses, which would later become a part of the Shell pavement design methodology (and that of other organizations as well).

A number of general solutions for determination of stresses and deformations in multilayer elastic solids also were presented at the 1962 conference. Additional related work was presented in 1967 at the Second International Conference. These general solutions, coupled with the rapidly advancing computer technology, fostered the development of the current generation of multilayer elastic and viscoelastic computer programs. Some of the most commonly used programs are given in Table 1. The widely used ELSYM program, developed at University of California at Berkeley by Ahlborn (*12*), most certainly benefitted from the 1962 and 1967 conference papers. [Although it never appeared in the published literature, the work of the Chevron researchers (*11*) must be acknowledged, because they presented the first computer solution for a five-layer system (CHEV5L) in 1963.]

In the late 1960s finite-element analyses to represent pavement response were developed by researchers such as Duncan et al. (*17*). Increasingly the finite-element method has been used to model pavement response, particularly to describe the nonlinear aspects of materials behavior. The significant work of Dehlen (*18*) and Hicks (*19*) illustrated how the nonlinear response of granular materials could be reasonably accounted for in pavement analyses. The program ILLIPAVE (*20*) incorporated many of the developments by Duncan et al. (*17*), Dehlen (*18*), and Hicks (*19*).

Current finite-element methodology has some advantages over layered-elastic and viscoelastic solutions because it provides greater flexibility in realistically modeling the nonlinear response characteristics of all the materials that make up the pavement section.

Although pavement engineers have recognized the importance of dynamic loading, it is only recently that such effects could be treated analytically. Lysmer and others (*21a*, *21b*) have developed a menu-driven program (SAPSI) for a

**TABLE 1 Examples of Multilayer Elastic Analysis Computer Programs**

| Program | Number of layers (max.) | Number of loads | Continuity conditions at interface | Probalistic considerations | Program source |
|---|---|---|---|---|---|
| BISAR (*10*) | 10 | 10 | full continuity to frictionless | no | Shell International Petroleum Co., Ltd., London, England |
| Remarks: | • Comparatively long running time since complete set of stresses and strains provided for each point. • Considers horizontal as well as vertical loads. | | | | |
| CHEV (*11*) | 5 | 2 | full continuity | no | Chevron Research Company |
| Remarks: | • Nonlinear response of granular materials accounted for in DAMA program of the Asphalt Institute which makes use of CHEV program. | | | | |
| ELSYM (*12*) | 10 | 100 | full continuity to frictionless | no | University of California at Berkeley |
| Remarks: | • Short running time for particular point. | | | | |
| PDMAP (PSAD) (*13*) | 5 | 2 | full continuity | yes | National Cooperative Highway Research Program (Project 1-10B) |
| Remarks: | • Running time is long for degrees of reliability other than 50-percent (the deterministic mode). • Iterative process used to arrive at moduli for untreated granular materials. | | | | |
| VESYS (*14*) | 5 | 2 | full continuity | yes | FHWA-US DOT |
| Remarks: | • Running time is long in probabilistic mode. • Program considers materials both as time independent (elastic) and time dependent (viscoelastic). | | | | |
| CHEVIT (*15*) | 5 | 12 | full continuity | yes | U.S. Army CE Waterways Experiment Station |
| Remarks: | • Modification of CHEV program. • Includes provision for stress sensitivity of granular layers. | | | | |
| CIRCLY (*16*) | 5+ | 10+ | full continuity to frictionless | no | MINCAB Systems, Canterbury, Australia (for Australian Road Research Board) |
| Remarks: | • Permits consideration of horizontal and vertical loads; in particular permits consideration of radially directed horizontal forces. • Can consider orthropic material behavior. • Permits consideration of strain energy. | | | | |

viscoelastic-layered system subjected to dynamic loads. This program may be run on a microcomputer and incorporates material properties that can be varied with the excitation frequencies of the load. The SAPSI program also permits the calculation of dissipated energy at any point in the asphalt-bound layer. This capability may prove extremely useful in the prediction of fatigue response, as will be seen in the discussion of the SHRP program research activities.

With improved microcomputer capabilities, it is now possible to simulate materials response to both load and environmental factors more accurately. The use of finite-element idealizations need not be limited to research applications. Rather they should be an integral part of routine pavement analysis and design.

## MATERIALS CHARACTERIZATION

An integral part of the development of analytically based methodologies has been the evolution of procedures to define requisite materials characteristics. For example, the solutions for stresses, strains, and deflections on which the analyses discussed in the previous section depend require a measure of the elastic or viscoelastic properties of the pavement materials. The solutions summarized in Table 1 are based on the assumption of *linear* response. [In some of the solutions, for example, PDMAP (PSAD) (*13*), an ad hoc representation of the nonlinear response of granular materials is utilized.] In reality, the majority of the materials used in pavement structures do not satisfy such an assumption. Accordingly, ad hoc simplifications of materials response must be used.

To define pavement materials response characteristics properly, one must consider the following service conditions: stress state (associated with loading), environmental conditions (moisture and temperature), and construction conditions (including water content and dry density for untreated materials). To ensure that materials evaluation is accomplished with reasonable cost, these service conditions must be carefully selected for use in laboratory testing.

Many of the existing pavement design procedures do not provide materials parameters in a form that can be used in the analysis methodologies described earlier. Certainly the California bearing ratio (CBR), the stabilometer *R*- and *S*-values, and the Marshall stability are not compatible with the analytically based procedures. Fortunately, there have been developments that have remedied these shortcomings. Although these developments have required simplifications, they generally have passed the test of engineering reasonableness, at least in my judgment.

One such simplification has been to describe the materials response in terms of the relationship between applied stress and recoverable strain measured in a repeated load test. Introduced by Seed (*22*), the term *resilient modulus* was used to describe this relationship and was based on his work with fine-grained soils. It has had, I believe, a significant impact on the development of analytically based design procedures and has been incorporated into the 1986 version of the *AASHTO Guide for Design of Pavement Structures* (*23*).

Research results of Seed and his colleagues (and of my colleagues as well), together with research on the resilient and dynamic stiffness properties of other paving materials, will be discussed in this section. Although this discussion focuses primarily on developments at the University of California at Berkeley, it also includes research by other organizations.

To determine the stresses, strains, and deflections in the various systems described above, it is necessary to have a measure of the stiffness characteristics of the various pavement components.

Although there has been considerable research on the stiffness characteristics of fine-grained soils and granular materials, impetus for these efforts was provided by the research of Seed and his associates, particularly Chan, Lee, and Mitry (*22, 24, 25*). In addition, the research endeavors of Hicks (*19*) and Dehlen (*18*), which followed from those efforts, have supplied very useful information for design purposes.

The research at Berkeley by Secor (*26*), Alexander (*27*), Terrel (*28*), Epps (*29*), Sousa (*30*), and Tayebali (*31*) has provided useful information on the stiffness characteristics of asphalt-aggregate mixtures.

Similarly the studies on cement-stabilized materials by Mitchell and those associated with him, including Fossberg (*32*), Shen (*33*), and Wang (*34*), as well as the work of Pretorius (*35*) and Raad (*36*), have provided useful data on the stiffness characteristics of those materials as well.

### Fine-Grained Soils

The stiffness characteristics of fine-grained soils are dependent on dry density, water content, soil structure, and stress level. For a particular condition, the resilient modulus $M_R$ (one measure of stiffness) is dependent on the applied stress; that is,

$$M_R = F(\sigma_d) \quad (1)$$

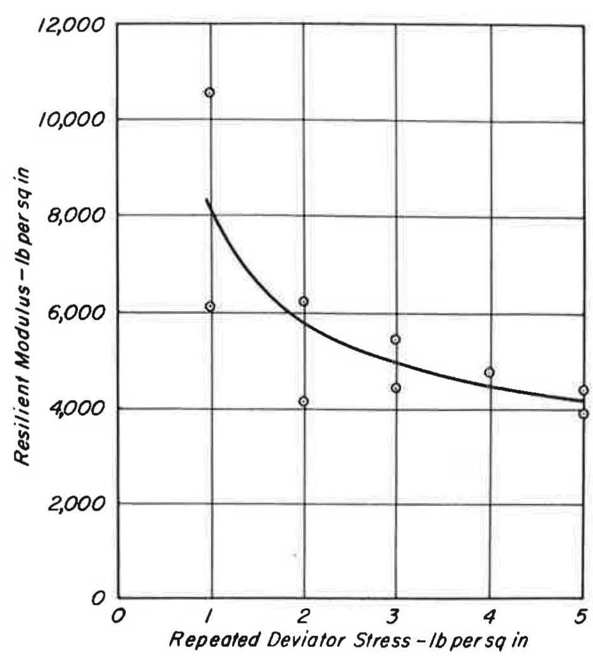


**FIGURE 2 Results of repeated load test, subgrade soil.**

where $F(\ )$ represents function of and $\sigma_d$ is the repeatedly applied deviator stress in a triaxial compression test.

Figure 2 provides an example of this dependency, and Figure 3 shows, for a particular stress state and number of load repetitions, the dependence of $M_R$ on water content and dry density (and presumably soil structure as well) for a wide range of potential service conditions.

For partially saturated soils (a condition representative of many subgrades worldwide), stiffness is dependent on the negative pore-water pressure (soil moisture section) as shown in Figure 4 (*18*). Data shown in Figure 5 suggest that laboratory-prepared specimens exhibit essentially the same stiffness as "undisturbed" specimens for comparable suction values (*18*).

Freeze-thaw action also influences the stiffness of fine-grained soils. When the soil is frozen, its stiffness increases; when thawing occurs, the stiffness is reduced substantially, as shown in Figure 6 (*37*), even though its water content may remain constant. This was originally suggested by Sauer (*38*). Such variations should be incorporated into the design process where appropriate.

To ensure that fine-grained soils tested in the laboratory for pavement design purposes are properly conditioned requires an understanding of soil compaction, particularly the relationship among water content, dry density, soil structure, and method of compaction (*39*). At water contents dry of optimum for a particular compactive effort, clay particles are arranged in a random array termed a "flocculated" structure.

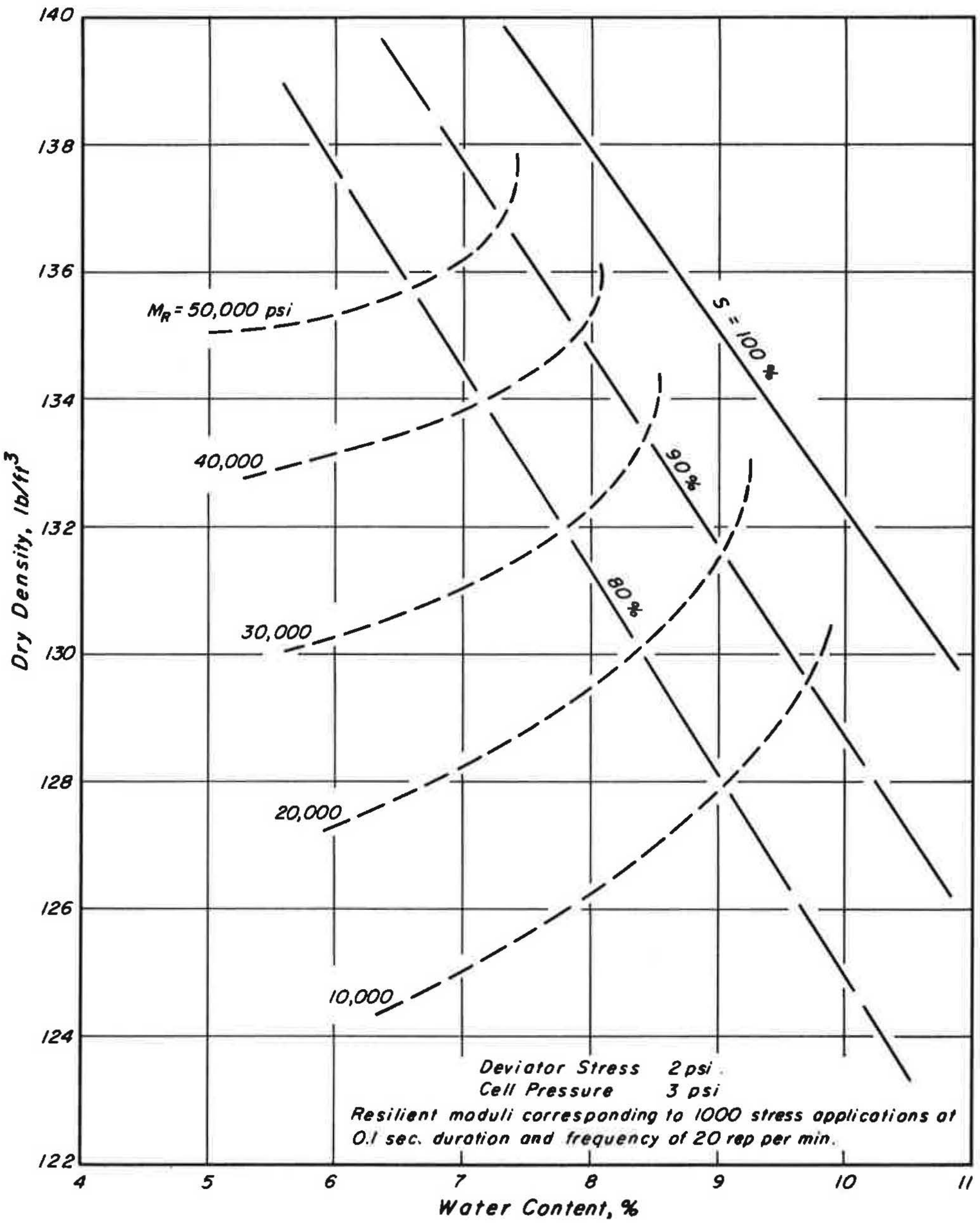


**FIGURE 3 Water content–dry density–modulus relationship for subgrade soil.**

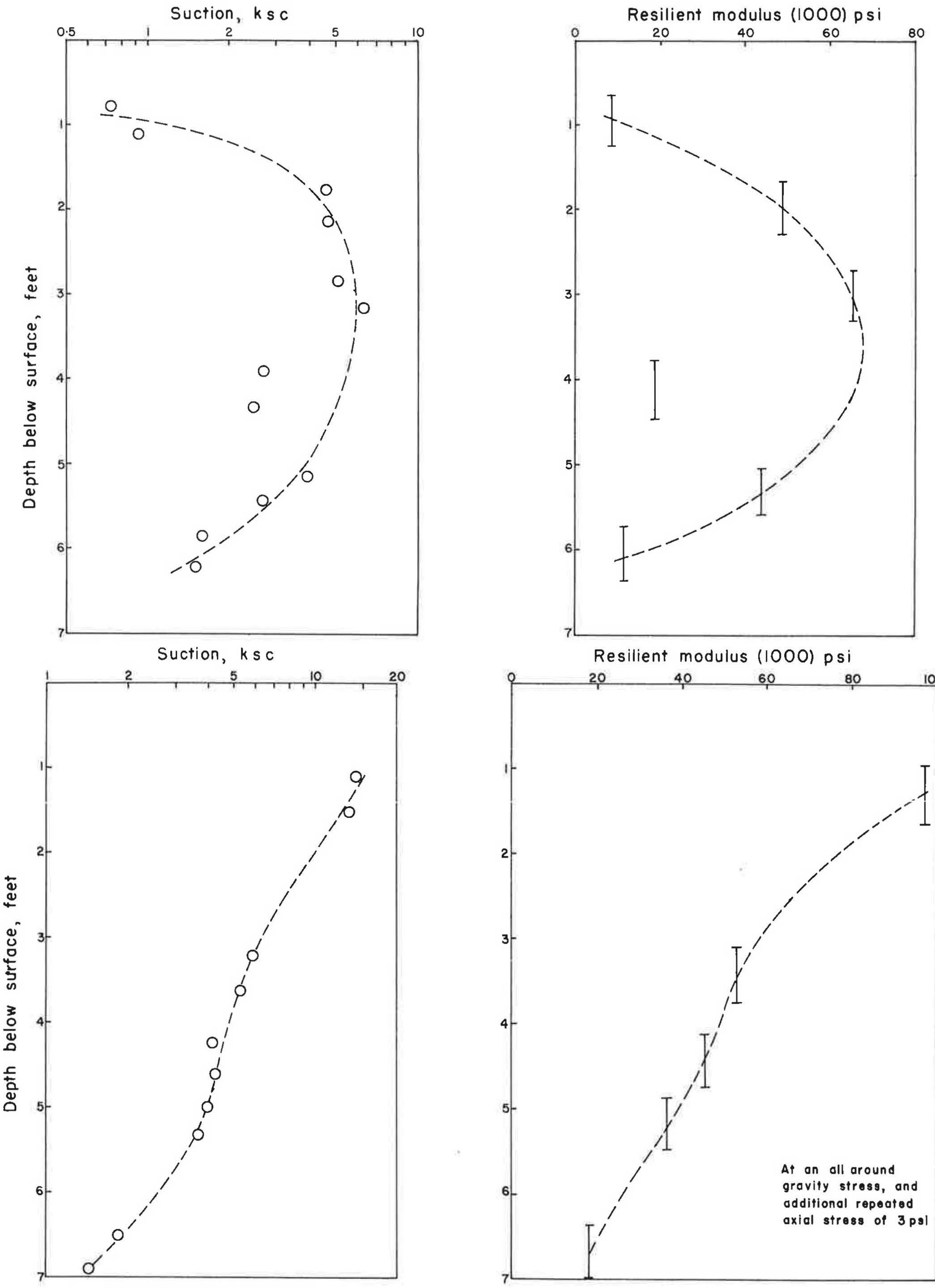


**FIGURE 4 Relationships between suction and resilient modulus (*18*).**

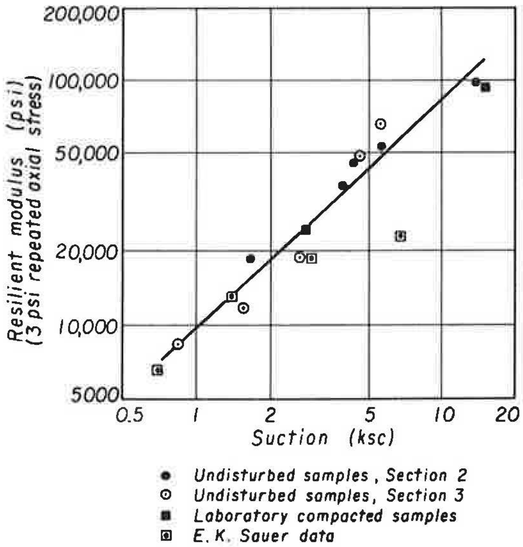


**FIGURE 5 Relationship between resilient modulus and suction, subgrade soil (*18*).**

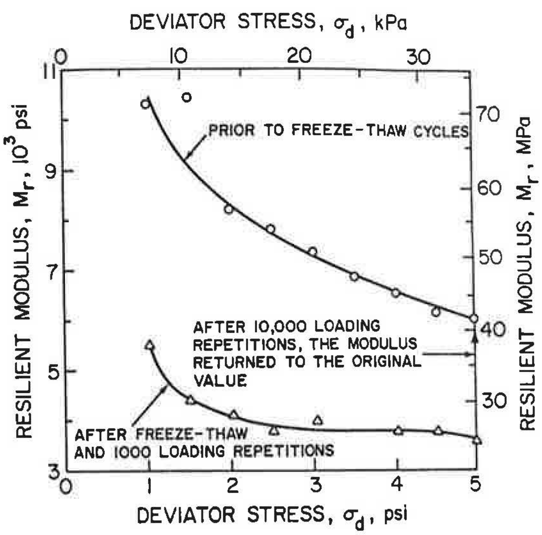


**FIGURE 6 Resilient modulus test results before and after freeze-thaw for undisturbed Regina clay (*37*).**

At water contents wet of optimum (provided shearing deformation is induced during compaction), particles are oriented in a parallel fashion, often termed "dispersed." These dispersed and flocculated compacted soil structures can lead to significant differences in mechanical properties for specimens assumed to be at the same water content and dry density.

To illustrate, consider a sample prepared by kneading compaction and soaked to a condition representative of that expected at some time subsequent to placement. The resilient response is shown in Figure 7 (H. B. Seed, unpublished data). If the designer were to compact the sample to the same initial condition by kneading compaction to save time in the laboratory (since it takes considerable time for a fine-grained material to become saturated), a different result would be ob-

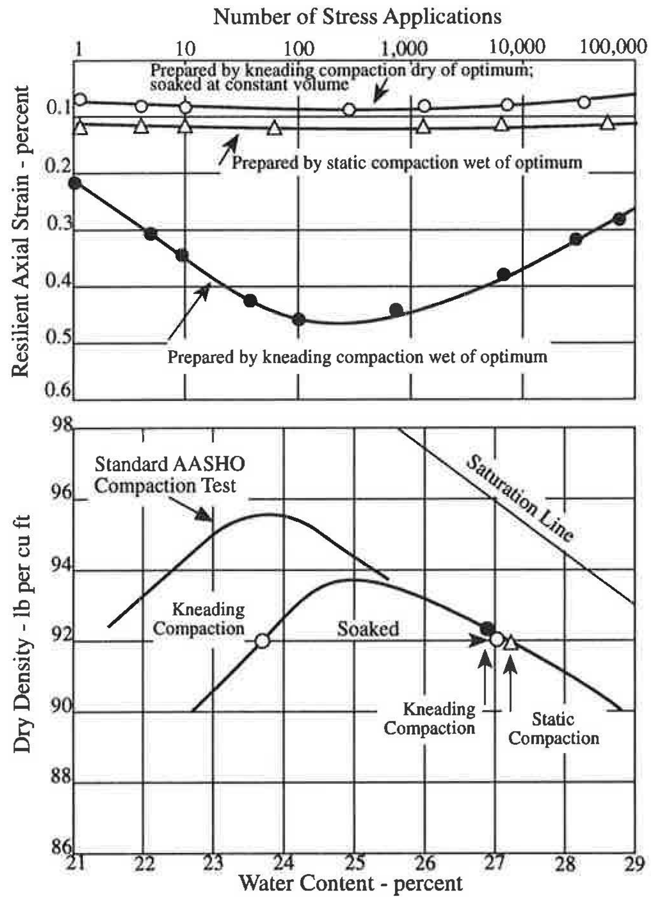


**FIGURE 7 Influence of soil structure on resilient response (H.B. Seed, unpublished data).**

tained. On the other hand, if the soil were prepared by static compaction to the same condition, essentially the same result would be obtained as for the situation in which the sample is prepared "dry" by kneading compaction and soaked to the particular state. In this case, static compaction wet of the line of optimums creates essentially the same structure as kneading dry of the line of optimums.

Thus, it is important that the designer understand these principles and utilize them in the selection of conditions for specimen preparation for testing. Guidelines based on such considerations are available (*40*).

### Untreated Granular Materials

The stiffness characteristics of untreated granular materials are dependent on the applied stresses. This stress dependency can be expressed in several different ways (*18, 19, 25, 41a, 41b*):

$$M_R = K\sigma_3^n \tag{2}$$

$$M_R = k_1\theta^{k_2} \tag{3}$$

$$M_R = F(p,q) \tag{4}$$

where

$\sigma_d$, $\sigma_3$ = deviator stress and confining pressure in a triaxial compression test, respectively;
$\theta$ = sum of principal stresses, in triaxial compression ($\sigma_d + 3\sigma_3$);
$q$ = $\sigma_d$ in a triaxial compression test;
$p$ = mean normal stress $(\sigma_d + 3\sigma_3)/3$; and
$K,n,k_1,k_2$ = experimentally determined coefficients.

The work of Dehlen, in which a granular material was characterized as a non-linear-elastic material, suggests that Equation 3 is a reasonable way to represent materials response. This form can be used in an ad hoc manner in layered elastic analyses (*42, 13*) and in finite-element idealizations (*17*). The work of Hicks (*19*) clearly demonstrated the efficacy of this latter approach.

Figure 8 shows the response represented by Equation 3. Table 2 contains a summary of aggregate responses representative of the behavior depicted by Equation 3 and Table 3, a summary of design moduli used by some agencies (*44*).

Although method of compaction is important for fine-grained soils because of soil structure considerations, the primary factors affecting the stiffness characteristics of granular materials are water content (degree of saturation) and dry density (*18, 19, 45*). Accordingly, any method of compaction (e.g., vibratory) that produces the desired dry density is considered suitable for laboratory preparation of specimens for testing.

**Asphalt-Aggregate Mixtures**

The stiffness characteristics of asphalt-aggregate mixtures are dependent on the time of loading and temperature:

$$S_{\text{mix}} = \frac{\sigma}{\varepsilon}\,(t,T) \tag{5}$$

where

$S_{\text{mix}}$ = mixture stiffness;
$\sigma,\varepsilon$ = stress and strain, respectively;
$t$ = time of loading; and
$T$ = temperature.

At temperatures above 25°C it is likely that the stress state has an influence on the stiffness characteristics of these materials, becoming more pronounced as the binder becomes less stiff. Figure 9 shows the dependence of mixture stiffness on both time of loading and temperature, and Figure 10 provides an indication of the range in stiffness that might be expected under moving wheel loads in different environments in the United States (*46*).

For some engineering applications, asphalt-aggregate mixtures can be treated as linear-viscoelastic materials, as demonstrated by Secor (*26*). The interchangeability of time and temperature can be considered applicable (i.e., the material can be assumed to be rheologically simple) as shown by Alex-

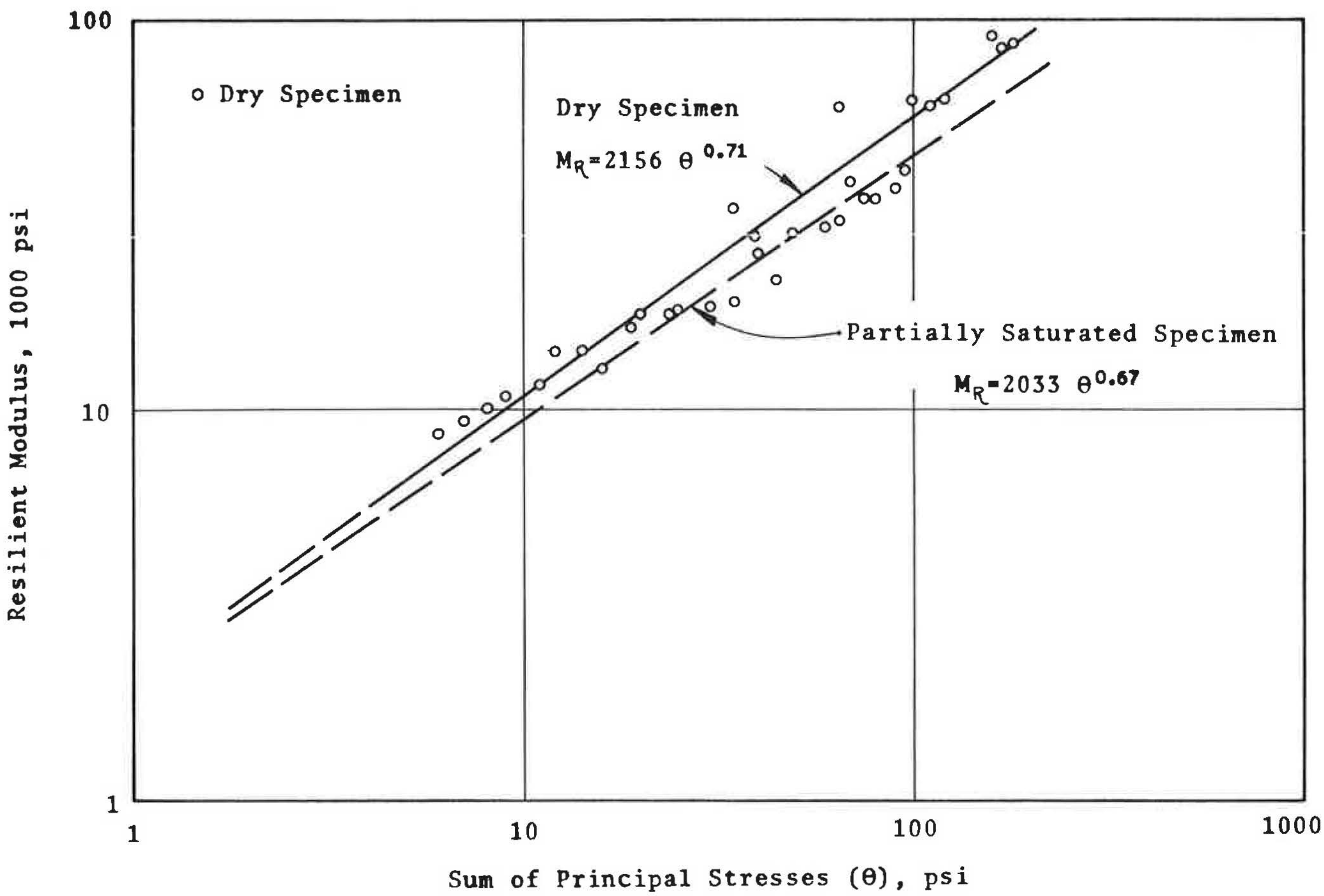


**FIGURE 8 Resilient modulus versus sum of principal stresses (σ), untreated aggregate base (*19*).**

**TABLE 2 Summary of Representative Repeated Load Triaxial Compression Test Data for Untreated Granular Materials**

| Material | $K_1$ | $K_2$ | Reference |
|---|---|---|---|
| Partially crushed gravel; crushed rock | 1,600 — 5,000 | 0.57 — 0.73 | (*19*) |
| Crushed stone | 4,000 — 9,000 | 0.46 — 0.64 | (*43*) |
| Well-graded crushed limestone | 8,000 | 0.67 | (*41*) |

ander (*27*) and others (*47–49*). For example, when the propensity of a mix for fatigue cracking is evaluated, mix stiffness used to calculate the critical stresses or strains can be determined using the above assumptions at a specific temperature and at a rate of loading corresponding to that of a rapidly moving vehicle (e.g., 0.02 sec). Use of this approach has been discussed elsewhere (*46*).

For permanent deformation evaluation, particularly at temperatures of 40°C or above, mixtures can no longer be treated as linear-viscoelastic. Under these circumstances research suggests that the material should be defined by nonlinear materials characteristics in order to properly model observed behavior like stress hardening and dilatancy (*50*). One approach by which this can be accomplished is shown in Figure

**TABLE 3 Representative Moduli for Untreated Granular Materials**

| Organization | Material | Modulus, ksi | |
|---|---|---|---|
| Belgium | Stone base | 72.5 | |
| | Subbase | 29.0 | |
| Czechoslovakia | Subbase | 21.8 | |
| Italy | Granular material | 36.3 | |
| U.S.A., FHWA | AASHO Base: | | |
| | Spring | 30.0 | |
| | Other seasons | 40.0 | |
| South Africa, NITRR | AASHO Subbase: | | |
| | Spring | 15.0 | |
| | Other | 20.0 | |
| | Overlaying Cement Stabilized Layer | | |
| | High quality crushed stone | Range | 36 - 130 |
| | | Design value | 65 |
| | Crushed stone/natural gravel | Range | 29 - 116 |
| | | Design value | 51 |
| | Gravel base | Range | 25 - 102 |
| | | Design value | 51 |
| | Gravel subbase | Range | 21 - 65 |
| | | Design value | 36 |
| | Overlaying Untreated or Cracked Stabilized Layer | | |
| | High quality crushed stone | Range | 25 - 87 |
| | | Design value | 29[a] |
| | Crushed stone/natural gravel | Range | 15 - 65 |
| | | Design value | 29[a] |
| | Gravel base | Range | 15 - 65 |
| | | Design value | 29[a] |
| | Gravel subbase | Range | 11 - 58 |
| | | Design value | 29[a] |
| | Gravel subbase (lower quality) | Range | 7 - 44 |
| | | Design value | 21[a] |

[a] The values shown are for bases or subbases under asphalt concrete. For base courses directly under surface treatments, higher values are used; e.g., in the case of the crushed stone base, a design modulus of 36 ksi (versus 29 ksi) is recommended [Reference (*44*)].

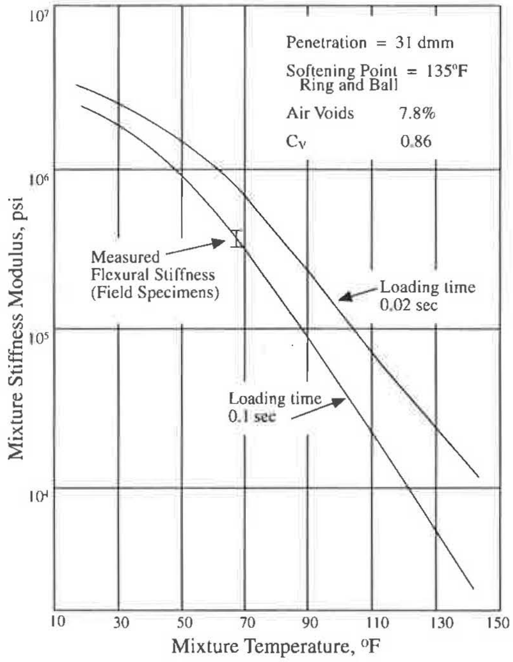


**FIGURE 9 Computed relationship between mix stiffness and asphalt concrete.**

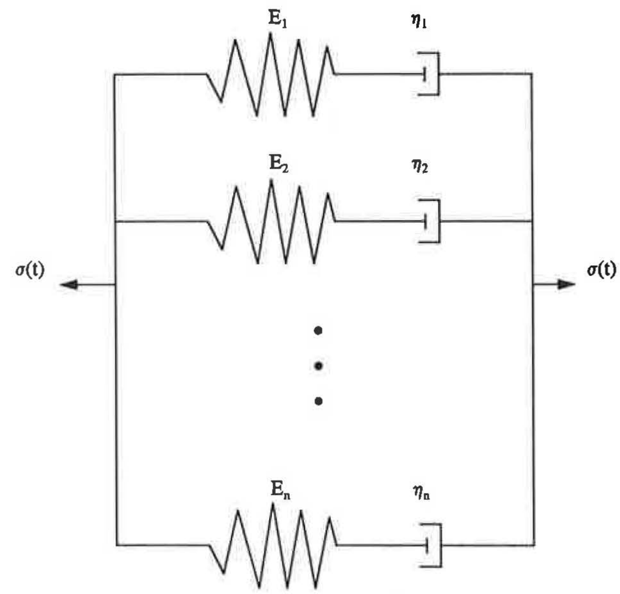


**FIGURE 11 Nonlinear viscoelastic representation for asphalt concrete.**

11, in which the material (asphalt concrete) is treated as a nonlinear-viscoelastic system containing at least five Maxwell elements. In this model nonlinear-elastic response characteristics (defined by the $E$'s) can be determined from a simple shear test (Figure 12). The linear-viscoelastic response (defined by the $\eta$'s in Figure 11) can be obtained from interpretations of the results of dynamic stiffness (or creep) testing over a range of frequencies (and temperatures).

When the response characteristics of an asphalt concrete mixture are measured to define its propensity for permanent deformation, it is important that the mixture be prepared by a compaction procedure that will reproduce an aggregate structure similar to that obtained in situ, including the effects of both the initial compaction process and repeated trafficking. Available evidence from Sousa et al. (*51*) and the Permanent International Association of Road Congresses (PIARC) (October 1990) suggests that a form of rolling wheel compaction is the method most likely to achieve these representative conditions.

In general, it is possible to define the stiffness characteristics of asphalt concrete in dynamic (sinusoidal), creep, or repeated loading. With today's equipment capabilities, dynamic loading over a range of frequencies (and temperatures) is the recommended procedure. If repeated loading in the diametral mode is used, it is suggested that the test be performed at

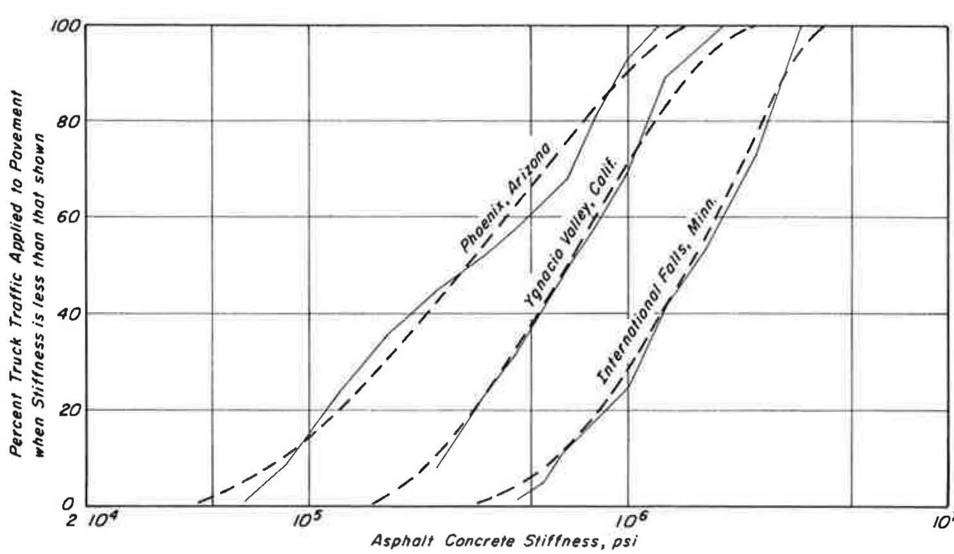


**FIGURE 10 Stiffness variation with traffic application, 12-in.-thick asphalt concrete layer.**

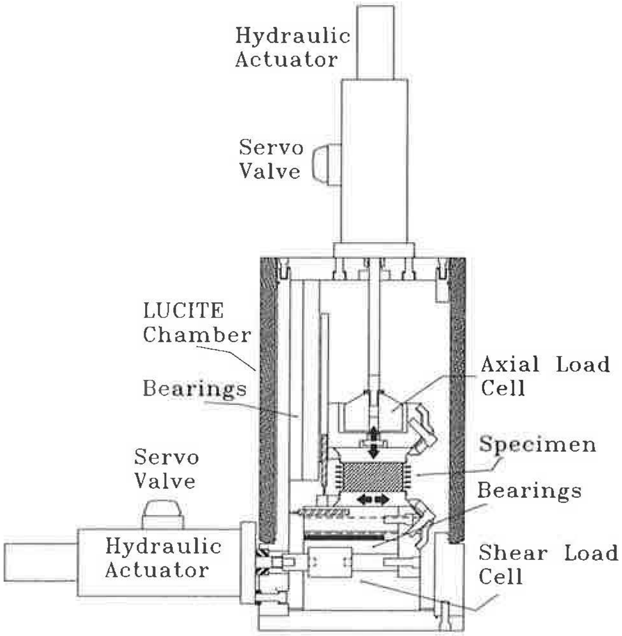


**FIGURE 12 Schematic representation of simple shear test equipment.**

temperatures equal to or less than about 25°C with only the horizontal deformations being measured to define the stiffness modulus (*52*).

### Asphalt-Emulsion-Treated Aggregate Mixtures

For aggregate mixtures treated with asphalt emulsion, the influence of curing on stiffness also must be evaluated. Terrel has presented a procedure whereby the influence of curing could be evaluated in the design process (*28*). Other investigators have incorporated these considerations into formal pavement design methodologies (*53, 54*). For example, both the Chevron and the Asphalt Institute procedures include a provision for damage development in the early life of pavements containing emulsified asphalt bases that are partially cured.

### Portland-Cement-Stabilized Mixtures

Studies of the stiffness characteristics of portland-cement-stabilized materials (*32–36, 55, 56*) indicate that stiffness moduli may range from approximately 10,000 to several million pounds per square inch, depending on soil type, treatment level, curing time, water content, and test conditions.

The research of Mitchell and his colleagues, some of which is summarized elsewhere (*55*), has developed useful relationships between the stiffness characteristics of cement-stabilized materials and various parameters such as unconfined compressive strength and flexural strength. These parameters have been used by other researchers as well and a summary of such values is included in Table 4 (*44*).

## ASPHALT CONCRETE DISTRESS CONSIDERATIONS

Three major modes of distress considered in the design of asphalt concrete pavements are *fatigue cracking, permanent deformation*, and *thermal cracking*. Distress criteria as well as prediction methodologies that can be used in the analysis and design process are briefly discussed for each of the three distress modes.

### Fatigue Cracking

When an asphalt-bound pavement layer is resting on an untreated aggregate base, the passage of a wheel load causes the pavement to deflect. The work of Hveem (*3*) demonstrated that the larger this deflection is and the higher the frequency of its occurrence, the greater the propensity for fatigue cracking. Hveem's work had, I'm certain, an impact on the development of laboratory fatigue tests on asphalt-aggregate mixtures to define this response. The resulting research has demonstrated that the fatigue response of asphalt concrete to repetitive loading can be defined by relationships of the following form (*46, 57*):

$$N = A\left(\frac{1}{\varepsilon_t}\right)^b \quad \text{or} \quad N = C\left(\frac{1}{\sigma_t}\right)^d \tag{6}$$

where

$N$ = number of repetitions to failure,
$\varepsilon_t$ = magnitude of the tensile strain repeatedly applied,
$\sigma_t$ = magnitude of the tensile stress repeatedly applied, and
$A, b, C, d$ = experimentally determined coefficients.

The work of Deacon (*58*), Epps (*29*), and others [such as that by Pell and his coworkers at Nottingham (*57, 59*)] has contributed significantly to our ability to design pavements to minimize fatigue cracking.

A design relationship utilized today by a number of organizations is based on strain and uses an equation of the form

$$N = K\left(\frac{1}{\varepsilon_t}\right)^a \left(\frac{1}{S_{\text{mix}}}\right)^b \tag{7}$$

which may involve a factor that recognizes the influence of asphalt content and degree of compaction and that is proportional to the following expression:

$$\frac{V_{\text{asp}}}{V_{\text{asp}} + V_{\text{air}}} \tag{8}$$

where $V_{\text{asp}}$ is the volume of asphalt and $V_{\text{air}}$ is the volume of air. Data developed by the researchers at Nottingham (*59*) and by Epps (*29*) have permitted the quantification of Equation 8, for example, in the Asphalt Institute design procedure (*60*).

Equation 7 is used in the Shell (*61, 62*) and Asphalt Institute (*63*) procedures with the coefficients set according to the amount

**TABLE 4 Stiffness Characteristics of Portland-Cement-Stabilized Materials**

| Source | Material | Unconfined Compression Stress at 7 days (psi) | Modulus of Rupture at 18 days (psi) | Stiffness Modulus (psi) | Poisson's Ratio |
|---|---|---|---|---|---|
| United Kingdom (dynamic loading) | Lean concrete | | 225 - 250 | 3.75 to 5.5 $\times$ $10^6$ | |
| | Cement-treated granular material | | 175 | 2.0 to 3.75 $\times$ $10^6$ | |
| | Soil cement | | 175 | 0.5 to 3.0 $\times$ $10^6$ | |
| Belgium | Lean concrete | | 175 | 2.2 $\times$ $10^6$ | |
| Netherlands | Sand cement | | 220 | | 0.25 |
| France | Cement-treated granular material | | | 2.9 $\times$ $10^6$ | 0.25 |
| | Sand cement | | | 1.75 $\times$ $10^6$ | 0.25 |
| South Africa | Pre-cracked phase | | | | |
| | Cement-treated: | | | | |
| | Crushed stone | 870 - 1740 | | 2.0 $\times$ $10^6$ | 0.35 |
| | Crushed gravel | 435 - 870 | | 1.2 $\times$ $10^6$ | 0.35 |
| | Gravel subbase | 110 - 215 | | 0.5 $\times$ $10^6$ | 0.35 |
| | Post-cracked phase a) under treated (bound layer) | | | | |
| | Crushed stone | | | 0.22 $\times$ $10^6$ | |
| | Crushed gravel | | | 0.14 $\times$ $10^6$ | |
| | Gravel subbase | | | 0.073 $\times$ $10^6$ | |
| | b) under untreated layer | | | | |
| | Crushed stone | | | 0.17 $\times$ $10^6$ | |
| | Crushed gravel | | | 0.11 $\times$ $10^6$ | |
| | Gravel subbase | | | 0.044 $\times$ $10^6$ | |

of cracking considered tolerable, the type of mixture that might be used, and the thickness of the asphalt-bound layer.

In the pavement structure the asphalt mix is subjected to a range of strains caused by a range of both wheel loads and temperatures. To determine the response under these conditions requires a cumulative damage hypothesis. A reasonable hypothesis to use [as demonstrated by Deacon (*58*)] is the linear summation of cycle ratios (sometimes referred to as Miner's hypothesis), which is stated as

$$\sum_{i=1}^{n} \frac{n_i}{N_i} \leq 1 \tag{9}$$

where $n_i$ is the number of actual traffic load applications at strain level $i$ and $N_i$ is the number of allowable traffic load applications to failure at strain level $i$. This equation indicates that fatigue life prediction for the range of loads and temperatures anticipated becomes a determination of the total number of applications at which the sum reaches unity.

Thus, prediction of fatigue life requires that tensile strains on the underside of the asphalt-concrete layer be computed for the range of traffic loads and materials and environmental conditions anticipated [e.g., as shown in the paper by Monismith et al. presented at the Sixth International Conference on the Structural Design of Asphalt Pavements (*64*)]. With knowledge of the applied traffic for each of the above conditions, Equation 9 can be used to estimate the propensity of the particular structure for cracking.

An alternative approach is to utilize the concept of dissipated energy suggested by Chomton and Valayer (*65*) and van Dijk (*66*). Current work in the SHRP asphalt research program (*67*) suggests that the relationship presented by the aforementioned researchers is mix specific, potentially "capturing" the effects of mode of loading, rest periods, temperature, and specimen configuration test type (in flexure). That relationship is

$$W_D = AN^z \tag{10}$$

where

$W_D$ = total dissipated energy to fatigue failure,
$N$ = number of load repetitions to failure, and
$A, z$ = experimentally determined coefficients.

Moreover, it is possible to compute the dissipated energy in a pavement structure by using a program like SAPSI (*21a, 21b*). Thus it is possible to check the adequacy of a specific pavement section (and asphalt mixture) if an estimate can be made of the total dissipated energy expended for a prescribed amount of traffic.

**Permanent Deformation and Rutting**

Rutting in paving materials develops gradually with increasing numbers of load applications, usually appearing as longitudinal depressions in the wheel paths accompanied by small upheavals to the sides. It is caused by a combination of densification (decrease in volume and hence increase in density) and shear deformation and may occur in any or all pavement layers, including the subgrade. Available information suggests that shear deformation rather than densification is the primary rutting mechanism (*68*).

From a pavement design standpoint two approaches have evolved to consider rutting. In the first, the vertical compressive strain at the subgrade surface is limited to a value associated with a specific number of load repetitions, this strain being computed by means of a layered-elastic analysis. The logic of this approach, first suggested by the Shell researchers [e.g, see the paper by Dormon (*9*)], is based on the observation that, for materials used in the pavement structure, permanent strains are proportional to elastic strains. Limiting the elastic strain to some prescribed value will also limit the plastic strain. Integration of the permanent strains over the depth of the pavement section provides an indication of the rut depth. By controlling the magnitude of the elastic strain at the subgrade surface, the magnitude of the rut is controlled.

The second procedure attempts to predict the surface rutting resulting from contributions of permanent deformations in each of the pavement components. Although this procedure is appealing from an engineering standpoint, it is more complex than the first. Only this latter approach will be discussed here and the discussion will be limited to rutting estimations in the asphalt-bound layer.

A number of researchers [originally Barksdale (*69*) and then, for example, McLean (*70*) and Freeme (*71*)] have presented procedures that use elastic analyses to compute stresses within the asphalt-bound layer and constitutive relationships that relate the stresses so determined to permanent strain for specific numbers of stress repetitions (termed the layered-strain procedure). Integration or summation of these strains over the layer depth provides a measure of the rutting that could develop.

One version of this approach was developed by the Shell researchers and has been used in modified form to evaluate specific pavement sections (*61, 62*). In this methodology, creep test results are incorporated into the following expression to estimate rutting:

$$\Delta h_1 = C_M \sum_{i=1}^{n} \left[ h_{1-i} \cdot \frac{(\sigma_{\text{ave}})_{1-i}}{S_{\text{mix}_{1-i}}} \right] \tag{11}$$

where

$\Delta h_1$ = permanent deformation in the asphalt-bound layer,
$h_{1-i}$ = thickness of sublayer of asphalt-bound layer with thickness $h_1$,
$(\sigma_{\text{ave}})_{1-i}$ = average vertical stress in layer $h_{1-i}$, and
$S_{\text{mix}_{1-i}}$ = mix stiffness for layer $h_{1-i}$ for specific temperature and time of loading (obtained by summing the individual times of loading of the moving vehicles passing over that layer at the specific temperature).

Although this procedure is not sufficiently precise to predict the actual rutting profile due to repeated trafficking, it provides an indication of the relative performance of different mixes containing conventional asphalt cements. {In special projects [e.g., as described by Monismith et al. (*64*)], field documentation may provide a reasonable measure of $C_M$ for use in estimation of performance of pavements containing comparable mixes.} If it is planned to use mixes containing modified binders, creep test data within the framework of Equation 11 will not provide correct estimates of mix performance since mixtures containing these binders behave differently under loading representative of traffic as compared with their response in creep. Data suggest that the use of creep test data may overpredict rutting for mixes containing some modified binders.

To predict the development of rutting and the lateral movement that may lead to upheavals adjacent to the wheel tracks, a more complex analysis is required (*50*). One such methodology is being developed within the SHRP research program wherein the constitutive relationship is defined by a model like that shown in Figure 11 that combines non-linear-elastic response with linear-viscous response. This relationship is then used in a finite-element idealization [based on FEAP (*72*)] of the pavement system to permit calculation of the accumulation of permanent deformation with repeated trafficking. Such an approach has the potential for improved rutting prediction since it incorporates some of the mix factors that cannot be considered in the layer-strain procedure.

**Thermal Cracking**

Thermal cracking is a mode of distress generally manifested by transverse cracks at the pavement surface. In qualitative terms, cracking is developed as follows. As the temperature at the pavement surface drops, a temperature gradient develops through the depth of the layer, since time is required for the cold to be conducted into the system. The surface of the pavement attempts to contract, but this contraction is restrained by the lower portions of the pavement. Tensile stresses will develop because of this restraint. Initially the stresses are small since the stiffness of the mix is relatively low. However, as the temperature decreases, the tendency to deform increases, but the lower portions of the pavement still prevent the deformation from occurring. Mix stiffness is also increased at lower temperatures; this increased stiffness, coupled with the propensity for increased contraction, eventually leads to tensile stresses at the surface that exceed the fracture (tensile) strength of the asphalt concrete and result, in turn, in surface cracking (*73*).

Assessment of the low-temperature response of asphalt pavements thus requires a knowledge of mix stiffness characteristics at longer times of loading and the fracture (tensile) strength characteristics, both of which must be defined at the low temperatures expected in pavement structures.

Like mix stiffness, fracture strength is dependent on both time of loading and temperature. Both Epps (*29*) and Salam (*74*) have presented data on the tensile strength of asphalt-aggregate mixtures that support the significant work of Heukelom (*75*), which permits the tensile strength of mixes to be defined with minimal testing.

Although there are a number of approaches that address the prediction of the potential for low-temperature (thermal) cracking, that suggested by Christison et al. (*76*) is considered most suitable at this time since it accounts for the significant factors contributing to this mode of distress in a framework that is readily analyzed. In this procedure the asphalt concrete is considered as a pseudoelastic beam and the stresses resulting from the restraint of temperature-induced strains are determined from

$$\sigma_x(t) = \int_{t_0}^{t} S_{\text{mix}}(\Delta t, T)\alpha(T)dT(t) \quad (12)$$

where

$S_{\text{mix}}$ = mix stiffness dependent on $\Delta t$, $T$,
$\sigma_x(t)$ = thermal stress, and
$\alpha(T)$ = coefficient of thermal contraction.

With input of actual temperature data, thermal stresses can be determined near the surface of the asphalt-bound layer according to Equation 12. By comparing these stresses with the tensile strength of the asphalt concrete, the propensity for fracture can be determined. This is shown schematically in Figure 13 (*77*).

Within the SHRP research program, a direct determination of the fracture temperature has been developed. The Thermal Stress Restrained Specimen Test (TSRST) attempts to simulate conditions analogous to those represented by Equation 12. In the TSRST the thermal stress is developed at a specific rate of cooling as shown schematically in Figure 14 (*78*). When the thermal stress reaches the fracture strength of the mix (for the particular conditions), the specimen fractures. This should correspond to the development of thermal cracks in the pavement. With the TSRST it is possible to evaluate mixes in advance of construction by ensuring that the temperature at which fracture occurs in the test is less than that anticipated in situ.

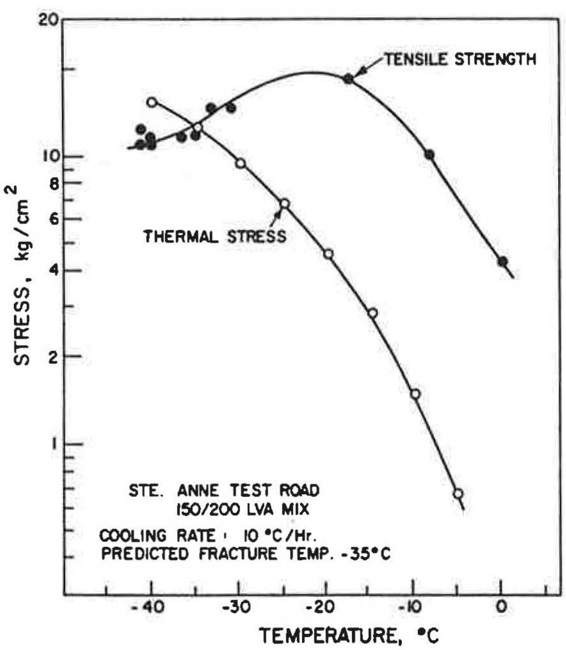


**FIGURE 13 Prediction of fracture temperature for a restrained strip of asphalt concrete (*77*).**

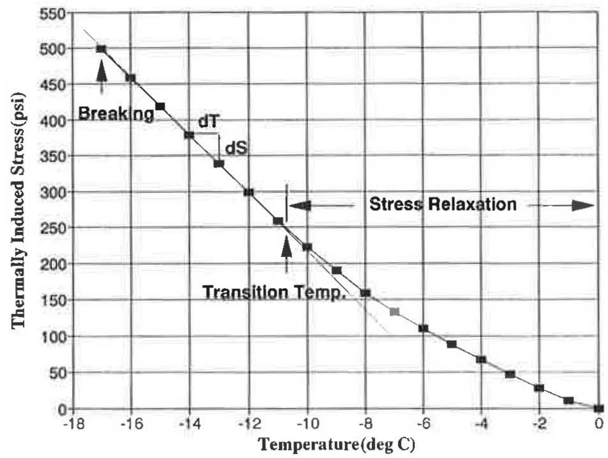


**FIGURE 14 Stress-temperature relationship in TSRST.**

## PAVEMENT DESIGN AND REHABILITATION

A number of pavement design and rehabilitation procedures using analytically based (mechanistic-empirical) methodologies are now in place, having received impetus from presentations and discussions at the First International Conference on the Structural Design of Asphalt Pavements in 1962 as noted earlier. These design procedures, some of which are briefly summarized in Table 5, use the type of research described in the previous sections. Generally, they follow the framework given in Figure 1 and have been utilized for the design of both highway and airfield pavements. Criteria used in some of these methodologies have been developed from analyses of the AASHO Road Test data [e.g., work by Finn et al. (*13*)]. Moreover, the results of the AASHO Road Test provided the data base against which to check the thickness-selection process associated with some of these methodologies. For example, the work by Witczak to validate the current Asphalt Institute procedure [described in Asphalt Institute Research Report 82-2 (*80*)] provides an excellent example of the use of this well-documented information.

### General Approach to Pavement Design

Analytically based design is possible because of the advances made in computer solutions used to represent pavement structures and developments in materials characterization that have evolved during the past 30 years. Although pavement re-

**TABLE 5 Examples of Analytically Based Design Procedures**

| Organization | Pavement representation | Distress modes | Environmental effects | Pavement materials | Design format |
|---|---|---|---|---|---|
| Shell International Petroleum Co., Ltd., London, England *(61,62,79)* | Multilayer elastic solid | Fatigue in treated layers<br>Rutting:<br>• subgrade strain<br>• estimate in asphalt bound layer | Temperature | Asphalt concrete, untreated aggregate cement-stabilized aggregate | Design charts; computer program: BISAR |
| Remarks: | Reference *(61,62)* is for highways. Procedure can be used for airfield pavements and BISAR is recommended for analyses of stresses and strains. | | | | |
| The Asphalt Institute, Lexington, KY (MS-1) *(60,63,80)* | Multilayer elastic solid | Fatigue in asphalt treated layers<br>Rutting:<br>• subgrade strain | Temperature, freezing and thawing | Asphalt concrete, asphalt emulsion treated bases, untreated aggregate | Design charts; computer program: DAMA |
| Remarks: | Applicable to highway pavements. | | | | |
| National Institute for Transport and Road Research (NITRR) South Africa *(81,82)* | Multilayer elastic solid | Fatigue in treated layers<br>Rutting:<br>• subgrade strain<br>• shear in granular layers | Temperature | Gap-graded asphalt mix, asphalt concrete, cement-stabilized aggregate, untreated aggregate | Catalogue of designs; computer program |
| Remarks: | Applicable to highway pavements | | | | |
| Federal Highway Administration U.S. DOT, Washington, D.C. *(84)* | Multilayer elastic or viscoelastic solid | Fatigue in treated layers<br>Rutting:<br>• estimate at surface<br>Serviceability (as measured by PSI) | Temperature | Asphalt concrete, cement-stabilized aggregate, untreated aggregate, sulphur-treated materials | Computer program: VESYS |
| Remarks: | Applicable to highway pavements. | | | | |
| University of Nottingham, Great Britain *(85,86)* | Multilayer elastic solid | Fatigue in treated layers<br>Rutting:<br>• subgrade strain | Temperature | Hot rolled asphalt (gap-graded mix), dense bituminous macadam (continuous graded asphalt concrete), untreated aggregate | Design charts; computer program |
| Remarks: | Applicable to highway pavements. | | | | |

***(continued on next page)***

sponse is typically computed using some form of elastic analysis, the material properties utilized must necessarily be tempered with engineering judgment since the materials that make up pavement structures do not exhibit linear-elastic behavior characteristics, as noted in previous sections.

Although specific design procedures are available, examples of which are shown in Table 5, it may be desirable for some projects, either highways or airfields, to consider a more general framework, such as that shown in Figure 15. Such an approach is not likely to be incorporated into a single computer program. Rather, it requires that the designer interact with the computer, making preliminary judgments regarding specific materials and thicknesses, performing some desk calculations to ensure that sections and conditions are representative, and then using a work station for detailed computations of stress, strain, and damage.

An example of this approach has been described by Monismith et al. (*64*), following the format in Figure 16. For the example, the computer program ELSA (an updated version of ELSYM5) was utilized. The necessity for a work station is apparent since computations are made for the complete state of stress and strain at several locations in the pavement section (Figure 17). These computations permit the evaluation of fatigue in the asphalt concrete, rutting at the pavement surface by controlling the subgrade strain, and permanent deformation within the asphalt-bound layer using the modification to the Shell procedure described earlier. For fatigue, aircraft wander could also be considered, as shown in Figure 18. If such computations were performed for a series of aircraft using the facility, damage at a number of locations transverse to the direction of traffic could be obtained by using Equation 9, permitting identification of the critical location.

TABLE 5 (*continued*)

| Organization | Pavement representation | Distress modes | Environmental effects | Pavement materials | Design format |
|---|---|---|---|---|---|
| Laboratoire Central de Ponts et Chaussees (LCPC), France (*87*) | Multilayer elastic solid | Fatigue in treated layers<br>Rutting | Temperature | Asphalt concrete, asphalt-treated bases, cement stabilized aggregates, untreated aggregates | Catalogue of designs; computer program |
| Remarks: | Applicable to highway pavements. | | | | |
| Centre de Recherches Routieres, Belgium (*88*) | Multilayer elastic solid | Fatigue in treated layers<br>Rutting | Temperature | Asphalt concrete, asphalt stabilized bases, untreated aggregates | Design charts; computer program |
| Remarks: | Applicable to highway pavements. | | | | |
| National Cooperative Highway Research Program (NCHRP) Project 1-10B Procedure (AASHTO) (*13*) | Multilayer elastic solid | Fatigue in treated layers<br>Rutting | Temperature | Asphalt concrete, asphalt stabilized bases, untreated aggregates | Design charts; computer program |
| Remarks: | Includes consideration of variability; applicable to highway pavements. | | | | |
| National Cooperative Highway Research Program (NCHRP) Project 1-26 Procedure (AASHTO) (*89*) | Finite element idealization<br>Multilayer elastic solid | Fatigue in asphalt concrete<br>Rutting:<br>• subgrade strain | Temperature | Asphalt concrete, untreated aggregates | ILLI-PAVE; elastic layer programs; e.g., ELSYM |
| Remarks: | Applicable to highway pavements. | | | | |

### Overlay Pavement Design

Like the design methodologies for new pavements using an analytically based approach, design procedures for overlay pavements have effectively made use of the research results of the last 30 years as well. A general framework for overlay design is presented in Figure 19 (*90*). The same types of analyses as those used for new pavements to define propensity for fatigue and rutting are also applicable to overlay design, that is, the use of elastic analysis and material characteristics of the type discussed earlier.

One of the key elements in overlay design is the nondestructive evaluation of the existing pavement (Figure 19). The development of the Benkelman beam has played a key role in the overlay design process and today still provides a comparatively inexpensive device for pavement evaluation. It should be noted, however, that the falling weight deflectometer (FWD) has become the most widely used device to define the characteristics of the existing pavement.

For overlay thicknesses to minimize fatigue, there are two alternatives, one in which the existing surface is considered intact and remaining life considerations allow selection of an overlay thickness. This thickness may be less than that for the second alternative, in which the existing asphalt concrete is no longer intact. In this latter situation a reduced stiffness of the existing asphalt-bound layer must be used.

For rutting, on the other hand, it can be assumed that the rut in the existing pavement, if any, will be filled. The thickness of the overlay will be that required to control the development of rutting and will only be a function of the additional traffic to be applied on the "new" pavement structure with the overlay. NCHRP Synthesis 116 (*90*) provides a summary of a number of the procedures utilized.

McCullough was one of the first to apply analytically based methodology to the design of asphalt concrete overlays on portland cement concrete pavements (*91*). Later, through finite-element analyses, Coetzee demonstrated the applicability of stress-absorbing membrane interlayers (SAMIs) to mitigate reflection cracking in asphalt concrete overlays on jointed (undoweled) portland cement concrete pavement (*92*).

## ASPHALT-AGGREGATE MIXTURE DESIGN

Asphalt mix design involves the selection of mix components and their relative proportions and requires a knowledge of the significant properties and performance characteristics of

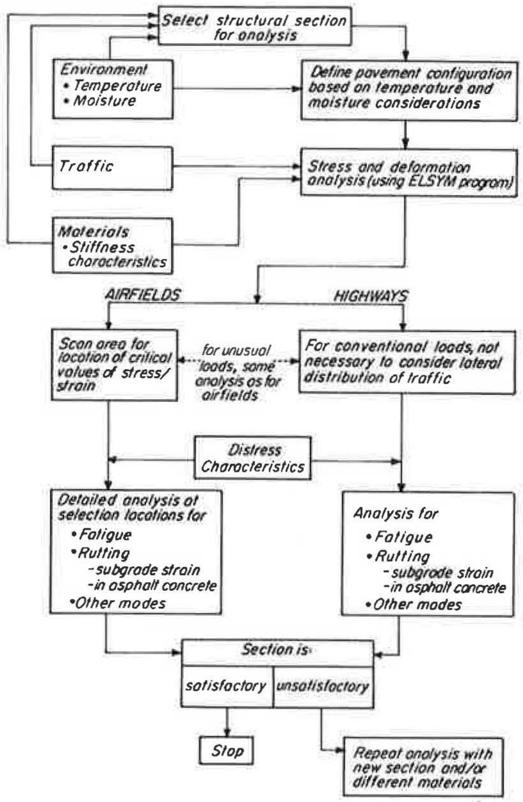


**FIGURE 15 General design methodology.**

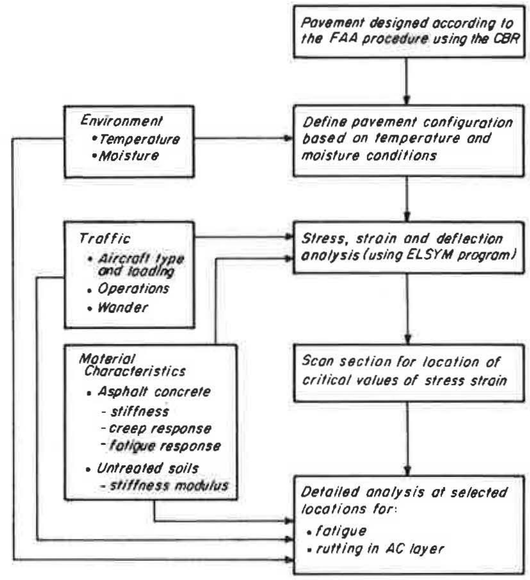


**FIGURE 16 Analysis procedure.**

asphalt paving mixtures and how they are influenced by the mix components.

It is desirable to select the components to achieve a reasonable balance in mix properties for the specific pavement application. Selection of the components and their relative proportions is also influenced by the pavement structural section into which the mix will be incorporated. Thus, the mix designer must be cognizant of the fact that mix design and pavement design are interrelated and must be considered together.

In general terms, mix design consists of the following basic steps: selection of the type and gradation of the aggregate; selection of the type and grade of the asphalt binder, with or without modification; and selection of the binder content. These steps have been incorporated into a general framework for design, as shown in Figure 20 (*93, 94*).

In the comprehensive design system of Figure 20, it will be noted that the system consists of a series of subsystems in which the mix components, asphalt (or binder) and aggregate, and their relative proportions are selected in a step-by-step procedure to produce a mix that can then be tested and evaluated to ensure that it will attain the desired level of performance in the specific pavement section in which it is to function. The influence of environmental factors, the effects of traffic loading, and the consequence of the pavement structural section design at the selected site are also included in this evaluation.

It is important to note in Figure 20 that an evaluation for water sensitivity of the mix is scheduled in the trial design

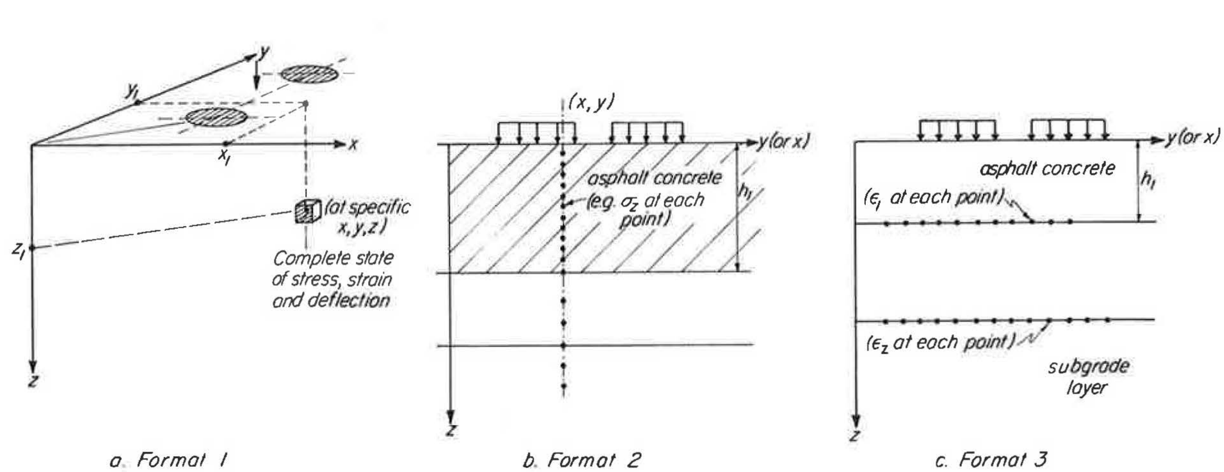


**FIGURE 17 Different formats for ELSA.**

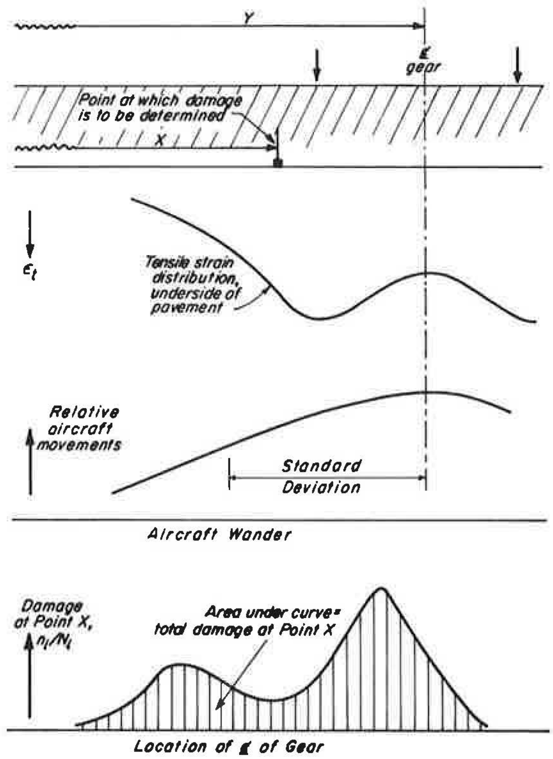


**FIGURE 18 Fatigue damage at a point on the underside of the pavement, taking aircraft wander into account.**

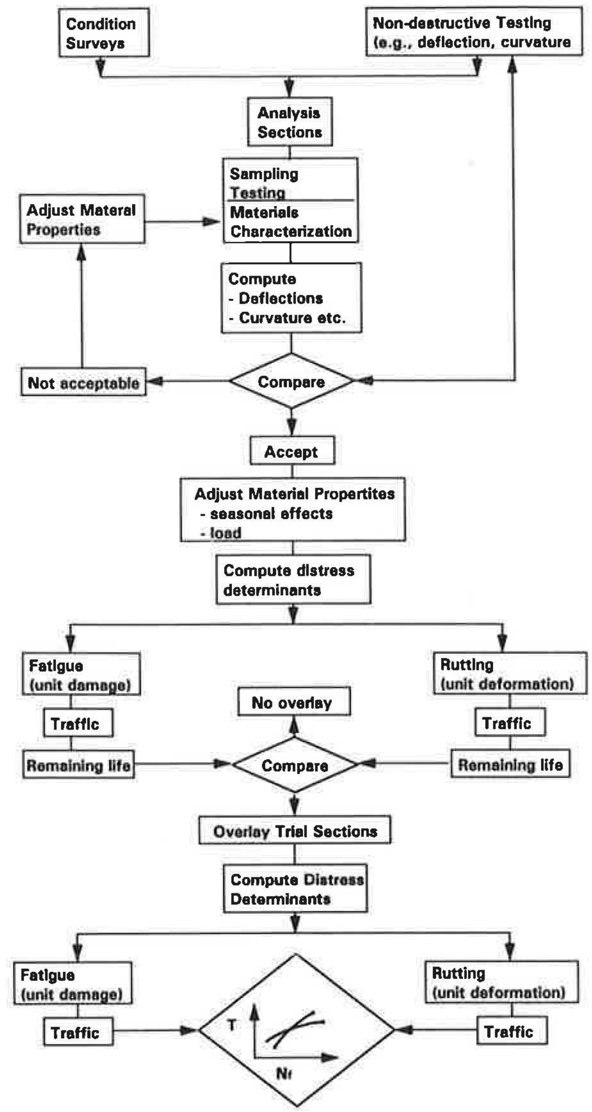


**FIGURE 19 Overlay design procedure using elastic layer theory to represent pavement response.**

subsystem. Satisfactory resolution of the water sensitivity problem before examination of the response of the trial mix to the three modes of distress shown in Figure 20 (fatigue, rutting, and thermal cracking) will allow full concentration on these evaluations.

Depending on the climatic conditions and loading factors to which the specific pavement is subjected, any or all of the distress modes may be evaluated. For example, in a hot, dry climate, it may not be necessary to examine the potential for thermal cracking, whereas because of the potential for fatigue, rutting, and surface raveling associated with reduced stiffness and asphalt embrittlement, it would be essential to evaluate these latter three modes.

Although satisfactory resolution of the water sensitivity problem is desirable in the initial design phase, it may not always be possible to completely preclude the deleterious influence of water or water vapor, or both. Accordingly, provision is also included in the distress evaluation phase for defining the characteristics of mixes that reasonably reflect the influence of this factor. In addition, the effects of long-term mixture aging must be considered. For example, as the mixture ages, its stiffness increases, leading in turn to increased propensity for thermal cracking. Both considerations are shown in Figure 20 as input at the appropriate point in the mixture-design process.

## IMPLICATIONS AND FUTURE DIRECTION

The developments discussed afford the engineering community an opportunity to expand its capabilities to improve both the design and rehabilitation procedures for heavily trafficked pavements.

The SHRP research program is developing improved procedures to measure the stiffness (as a function of both frequency-time and temperature), permanent deformation, fatigue, thermal cracking, aging, and water sensitivity characteristics of asphalt-aggregate mixes. Material properties determined from these tests can be used in performance-prediction models to estimate, as part of the design process and in advance of construction, the expected performance of the mix in a specific structural pavement section in a particular environment and for anticipated traffic loading.

Although some of the test procedures (including the method of compaction) appear to be more complex than those widely used today, it is my belief that we should not settle for something simplistic in the interest of expediency. It is important that technical merit rather than simplicity and ease of implementation drive the selection of test methods and analysis techniques. In the past 20 years we have all too often chosen

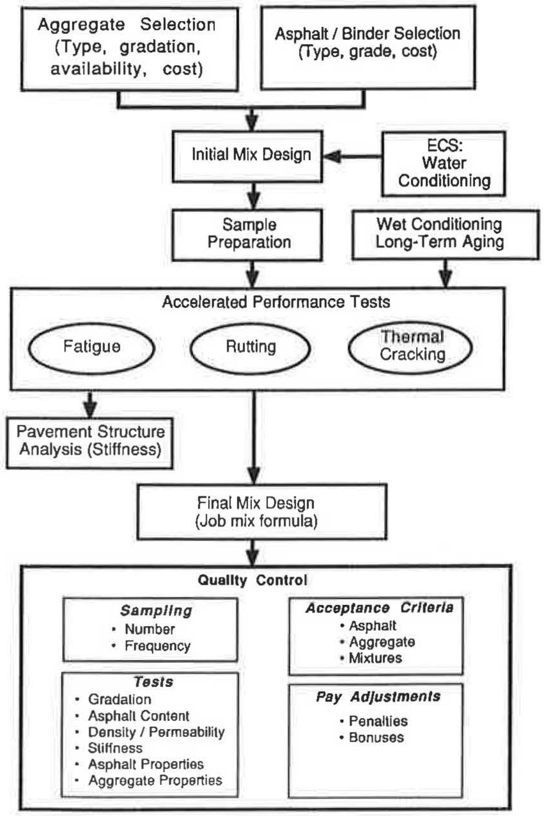


**FIGURE 20 Comprehensive mix design and analysis system.**

the latter course and, in effect, have let implementation considerations drive the selection process. Moreover, at times it has driven the research as well. Although it appears that we may be taking a chance, it really is not that much of a gamble and the results to be gained certainly appear worthwhile (e.g., improved performance and reliability). As Bernstein has recently stated,

> While our university research capabilities are unmatched by any other country, we have been unable to commercialize or capitalize on that strength. Other countries are quick to monitor our research activities, invest in them, take the results, and quickly move from the laboratory to the marketplace while we are still evaluating the risk and costs associated with trying something new. (*95*)

The comments from the participants in the 1990 European asphalt study tour (June 1991) clearly illustrate the point made in the previous paragraph. The group appeared to be enamored with European technology. In reality, the Europeans are implementing what was available in both the United States and Europe at about the same time. The Europeans in the transportation community have chosen to implement the technology, whereas those in the United States have not. Highway engineers and managers, including some of the those who were a part of the study tour, are responsible for this disparity. Let us not make the same mistake with the SHRP program: we must not permit simplicity and ease of implementation to dictate the outcome of the SHRP endeavor.

In the field of asphalt technology we must get away from what is sometimes termed the "Marshall" mindset. One way of doing this initially is to develop regional test centers where the new test methodologies can be implemented and some applied research can be accomplished as well. [The Laboratoire Central des Ponts et Chaussées (LCPC) has such a system in place in France.] This would allow the user agencies to take advantage of economies of scale and the pooled funding of resources. Until greater confidence in the new methodologies is more widely established, those who wish to participate can have their mix and pavement designs accomplished at these centers by skilled and knowledgeable personnel and obtain training in these new methodologies as well.

Although the SHRP program, as discussed here, is related to the asphalt technology aspects, there are a number of other areas toward which pavement engineers should direct their efforts. Examples are given in the following paragraphs.

The interaction between the vehicle (e.g., truck, aircraft) and the pavement has considerable potential for payoff in improved pavement performance and vehicle (suspension) design, which can benefit the population as a whole through reduced costs associated with the movement of goods. Research directed toward improved (dynamic) analysis capabilities for pavements, vehicle suspension designs, parametric studies of truck and pavement variables, development of new and improved dynamic load measuring systems, and development of an instrumented truck-trailer as a device to measure pavement response for pavement management activities is indicative of the types of investigations that can be accomplished. Increased interactions among the pavement engineers, the truck (and bus) manufacturing industry, and aircraft design groups are a prerequisite to successful activities in this area. Assessments must be made of the influence of the magnitude and type of aircraft and truck loadings on pavements (including the effects of load magnitude, gear and axle configurations, and contact pressures). The Turner Truck Study (*96*) recently completed by TRB can be considered a forerunner of the more detailed types of studies implied by the two research activities just described.

The development of new materials as well as the effective utilization of recycled materials must be accompanied by improved characterization and prediction techniques to ensure their proper use. Cooperation between engineers and engineering mechanicians and materials scientists is critical to the successful completion of such activities. Examples of this approach have been included in the SHRP program.

Consideration should be given to new design and rehabilitation techniques. So long as the materials to be utilized can be characterized and the proposed systems analyzed, analyses like those described here can establish their feasibility.

An important step in ensuring implementation of new materials and construction-rehabilitation procedures is the use of accelerated load testing devices. Such devices have been used extensively by European engineers but are virtually nonexistent in the United States. The accelerated loading facility (ALF) is a large-scale example; however, smaller, more versatile and faster laboratory-scale models could also be developed in the United States to accelerate evaluation of asphalt mix performance. Such devices should be considered as an essential part of the regional centers described above and could be part of the applied research activity referred to earlier.

Finally the importance of continued observation and documentation of performance cannot be overemphasized. At the conclusion of the AASHO Road Test, the Highway Research Board (HRB) (now the Transportation Research Board) prepared guidelines for implementing such a program (*97*). Unfortunately, these recommendations were not followed, and it was not until the Long-Term Pavement Performance (LTPP) Program of SHRP was implemented that the HRB recommendations came to fruition. We must ensure that the activities instituted through the SHRP program continue if we are to continue to improve pavement technology.

I would like to conclude on a more philosophical note by drawing on observations by W. R. Hudson relative to the elements of successful research. They include the following: (*a*) an overall plan with top-level commitment and sufficient and continuous funding assured; (*b*) a research environment, including personnel and facilities, that fosters flexibility and innovation; (*c*) interaction between researchers and practitioners; and (*d*) dissemination and implementation of the research results.

My comments will be directed toward some of these elements. Naturally, there must be a person or persons with the desire and capabilities to solve the problem at hand. Top-level commitment and support must be provided and there must be continuity of funding, since innovation doesn't occur on schedule. In providing the funding there must be, on the part of the funding agency, a willingness to take some risk, since all research is not successful. Moreover, there must be freedom for innovation. An excellent example of this was provided by W. N. Carey, Jr., the former Executive Director of TRB. As project engineer for the AASHO Road Test, he carefully weighed the comments of various advisory groups. Although he often accepted their advice, he protected his researchers from costly and nonproductive suggestions, allowing the staff the freedom to explore their innovative ideas.

Finally, we must recognize the need for intermediate and long-term research. There has been in the last 20 years too much emphasis on poorly coordinated and short-term research, and, as noted earlier, although implementation is important, it should not drive the research as it seems to have done. Implementation will come if the benefits can be demonstrated.

In conclusion, I would like to acknowledge again both the U.S. and the international pavement research communities. Though not delineated in detail, the results of their research efforts permeate this discussion. The friendships developed with Peter Pell, Stephen Brown, Jacques Bonnot, W. Heukelom, L. Nijboer, A. Klomp, K. Wester, J. Verstraeten, Charles Freeme, Neil Walker, John Yeaman, as well as with others of the international community, are treasured.

I indeed have been fortunate to have been in a research environment where some of the requisite factors discussed here were present, including

- Outstanding colleagues—H. B. Seed, J. K. Mitchell, C. K. Chan, J. M. Duncan, R. Horonjeff, J. Lysmer, R. Taylor, and others;
- Outstanding students, as already enumerated;
- A community of outstanding practitioners in close proximity—F. N. Finn, W. A. Garrison, F. N. Hveem, L. E. Santucci, R. J. Schmidt, G. B. Sherman, J. Skog, B. A. Vallerga, and C. J. Van Til; and
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Please accept my thanks for the opportunity to present this lecture.

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## PART 2

# Developments in Flexible Pavement Design

# Part 2
# Foreword

Carpenter examines rutting data from the AASHO Road Test and discusses methods that can assist an agency in developing suitable procedures for using the 1986 AASHTO Design Guide without relying on the current insufficient modulus relationships for the asphalt concrete layer. Elliott examines the resilient modulus value used to represent the AASHO Road Test subgrade in the AASHTO flexible pavement design equation and finds that appropriate values for noncohesive soils need further study. Hossain et al. describe the 5-year performance of a porous pavement built to store storm water: the pavement is performing well and both the water infiltration rate and the storage capacity are still above the design values. Hajek et al. present the results of a field study that investigated the performance of five pavements incorporating open-graded drainage layers (OGDLs). They determined that drainage layers will perform only as part of a properly designed internal drainage system. Romero et al. present a description of the CEDEX Road Research Center test track in Spain and the test carried out there. Bonaquist describes several of the test programs conducted by FHWA using the Accelerated Loading Facility that demonstrate the mobility and flexibility of the equipment.

# Load Equivalency Factors and Rutting Rates: The AASHO Road Test

SAMUEL H. CARPENTER

Current needs in state department of transportation (DOT) pavement design and management programs require explicit information about the development of rutting in asphalt concrete mixtures. Many state DOTs are considering implementing the 1986 AASHTO *Pavement Design Guide*, and they require guidance in analyzing mix performance as a part of pavement design. Rutting data are examined from the AASHO Road Test sections built over cement-treated bases. The recorded rut depths are analyzed to derive relationships for the asphalt mixture used during the AASHO Road Test. Load equivalency factors are derived from the rutting data clearly showing that the AASHTO rigid pavement load equivalency factors more closely indicate the development of rutting under different axle weights and configurations. The fundamental properties describing rutting potential provide data useful in comparing rutting behavior of current mixes with that developed in the AASHO Road Test.

The AASHO Road Test, conducted from 1958 through 1960, produced the major pavement design procedure in use today in the United States. Load equivalency factors (LEFs) were derived from the test for use in describing relative damage produced by different wheel loads. The LEFs from the Road Test are based on the relative decrease in serviceability caused by an increase in axle load, or number of axles. These general LEFs have not been shown to be applicable for a specific distress element, such as rutting, and there is no guarantee that one specific distress is affected to the same degree as serviceability, which is primarily roughness related. If an individual distress is being investigated, its development under mixed traffic must be demonstrated using an LEF value derived specifically for that distress. Each distress will potentially have different LEF values.

There is a concerted effort to investigate load equivalency factors and extend the AASHO Road Test values using more technologically acceptable methods of determination, such as deflection, strain, or even stress matching. Uzan and Sidess (*1*) investigated the Road Test loops with deflection and stress-based relationships to extend the AASHTO LEFs to different axle configurations and derive LEFs as a function of total pavement condition, not only the initial condition, as is the case with the AASHTO LEFs. This development allows for a more reasonable determination of axle damage occurring at some time after the pavement has been built. The study by Hajek and Agarwal investigated several methods of calculating LEFs to investigate the axle weights for different wheel configurations that would produce the same damage as a standard single or tandem axle (*2*). They clearly showed an effect of wheel spacing on all procedures. These studies all investigated the general LEF as related to loss of serviceability, as was determined in the AASHO Road Test. Each investigation recommended that mechanistic studies be used to determine distress-specific LEF values. Kenis and Cobb used the VESYS-5 program to investigate the strain and deflection relationships for LEF values under different axle configurations (*3*). They clearly demonstrated a seasonal effect on the LEF values and showed that the season of failure can have an impact on the LEF for the pavement. These LEF comparisons, however, although based on mechanistically calculated response values, were developed for the entire pavement structure, and the rutting comparisons cannot be used in conjunction with a study of rutting in asphalt concrete. The Kenis study also used constant rutting model parameters for all load levels, which is questionable given recent laboratory studies (*4*).

This paper details an investigation of several sections of the AASHO Road Test that provide rutting information relative to the asphalt concrete mixture used at the AASHO Road Test. The AASHO information consists of rutting development as a function of axle repetitions, with several axle loads and configurations being included. This analysis provides sufficient information to establish material properties relevant to rutting in the AASHO asphalt concrete mix and to calculate LEFs specifically related to rutting development. The results indicate the degree of error when improper LEFs are used to evaluate field rutting performance of asphalt concrete mixtures, which could be quite large. The analysis provides a basis for comparisons with new mixes to establish performance levels relative to those of the AASHO Road Test. Although not specifically including parameters that may have changed since the AASHO Road Test, such as tire types and pressures, the results provide a beginning point based on true field performance to establish the basis for making comparisons.

## AASHO ROAD TEST SECTIONS

Three of the six loops at the AASHO Road Test contained cement-treated base sections with asphalt concrete surfacing—Loops 4, 5, and 6. Each loop contained two sections, and each section contained four different base thicknesses, as follows:

- LOOP 4: 18-kip single axle in inner lane, 32-kip tandem axle in outer lane. Sections 557 and 563: 3-in. asphalt surface, base thicknesses of 9, 7, 5, and 3 in. in Subsections 1, 2, 3, and 4, respectively.

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- LOOP 5: 22.4-kip single axle in inner lane, 40-kip tandem axle in outer lane. Sections 461 and 465: 3-in. asphalt surface, base thicknesses of 14.4, 11.1, 7.9, and 4.6 in. in Subsections 1, 2, 3, and 4, respectively.
- LOOP 6: 30-kip single axle in inner lane, 48-kip tandem axle in outer lane. Sections 281 and 289: 4-in. asphalt surface, base thicknesses of 16.1, 12.4, 8.6, and 4.9 in. in Subsections 1, 2, 3, and 4, respectively.

Rutting measurements on these sections provide data that can be attributed to the behavior of the asphalt concrete mixes rather than the stabilized base. This assumes that the cement-treated base does not fail and allow a general shear failure in the pavement structure. If rutting behavior can be established from these sections, any resulting behavioral descriptors should relate solely to the asphalt concrete mixture. These results would provide a baseline of AASHO material performance against which new materials can be compared.

As reported in Transportation Research Board Special Report 61E (*5*), the thinner bases exhibited failure. This was evident in the data, which will be presented shortly. For these reasons, Sections 3 and 4 were eliminated from the study.

## ANALYSIS PROCEDURE FOR RUTTING MODELS

### Models

The first requirement for this analysis is to use a model that can accept field and laboratory data and to develop similar material properties from the analysis. Although there are some highly complex theories, several phenomenological models describe general rutting behavior in asphalt concrete mixes and are suitable for laboratory and field studies. The following equation can be used to characterize either permanent strain or rut depth:

$$\varepsilon_p = AN^B \tag{1}$$

where

$\varepsilon_p$ = permanent deformation, strain, or rut depth;
$N$ = number of load repetitions; and
$A,B$ = material properties.

As developed by Khedr (*6*), the rate of rutting can be expressed by

$$\varepsilon_p/N = AN^{B'} \tag{2}$$

where $B'$ is equal to $B - 1$.

### Section Data

The rutting rate model is most applicable to the study being conducted here because it allows a determination to be easily made relative to general pavement failure, or the onset of unstable mixture performance, and has been shown to function equally well in the laboratory and field for establishing material properties (*7, 4, 6*). Figure 1 contains the permanent strain rate [(rut depth/thickness)/$N$] plots for Section 557 (18-kip single axle) of the AASHO Road Test, with all subsections shown.

The failure of the base is clearly evident in Sections 3 and 4 when the data take an abrupt upturn at approximately 50,000 axle repetitions. The thick base Sections 1 and 2 carried the axle load to 1 million repetitions with no evidence of failure. The same trend is evident in Figure 2 for Loop 6, inner lane, a 30-kip single-axle section. These data support the findings of the Road Test that indicated base failure in the thin sections.

To allow general comparisons of different thicknesses, the rut depth measurements were converted to permanent strain by dividing by the asphalt thickness. For thicknesses in the 3- to 4-in. range, this is acceptable. Comparisons of thicker sections (6 in. of asphalt concrete or more) raise questions, because the thicker sections may not develop significant rutting in the lower depths and these lower depths should probably not be included in an analysis to compare mix performance. However, there is no analytical procedure today to allow this separation. The data for both subsections in a loop were combined and a regression analysis was performed on the resulting data set for that section or lane. The $A$ and $B'$ coefficients resulting from this analysis are given in Table 1.

The $A$ and $B'$ coefficients from this analysis are mix parameters that define the rutting behavior of the asphalt concrete mixture used in the AASHO Road Test. The values from each loop are representative of mix performance under the stress state induced by the different axle configurations, the climate of the 2-year test period, and loadings present on the different lanes and loops. Differences in the coefficients are attributable to the loading conditions on each loop or lane and not to mix differences, because great care was taken in the Road Test to ensure material uniformity.

The averaged $A$ and $B'$ values for each loop were then used to calculate the rut depth accumulation curves for each loop and axle configuration using Equation 2. The calculated rut depth accumulation curves for a 3-in.-thick layer are shown in Figure 3 for tandem axles and in Figure 4 for single axles.

## DEVELOPMENT OF LEFs FOR RUTTING

To develop the LEFs, the rut depth accumulation curves for each axle type are required. An LEF is defined as the number of load repetitions of a standard axle [18-kip equivalent single-axle load (ESAL)] to produce a defined level of damage (rut depth) divided by the number of load repetitions of another axle load or configuration to produce that same level of damage.

In this analysis, rutting has been singled out as the sole variable to investigate for its relationship to axle loads. Damage levels considered as failure in a rutted pavement can vary between agencies, and for this evaluation three rut depths were selected for comparison: 0.3, 0.4, and 0.5 in. The load repetitions calculated for each axle load and configuration to produce the assigned level of rutting in a 3-in. pavement surface constructed with the AASHO Road Test asphalt concrete mixture are shown in Table 2.

With the 18-kip ESAL as the standard and the LEF calculated for each axle, the LEFs for rutting were obtained by

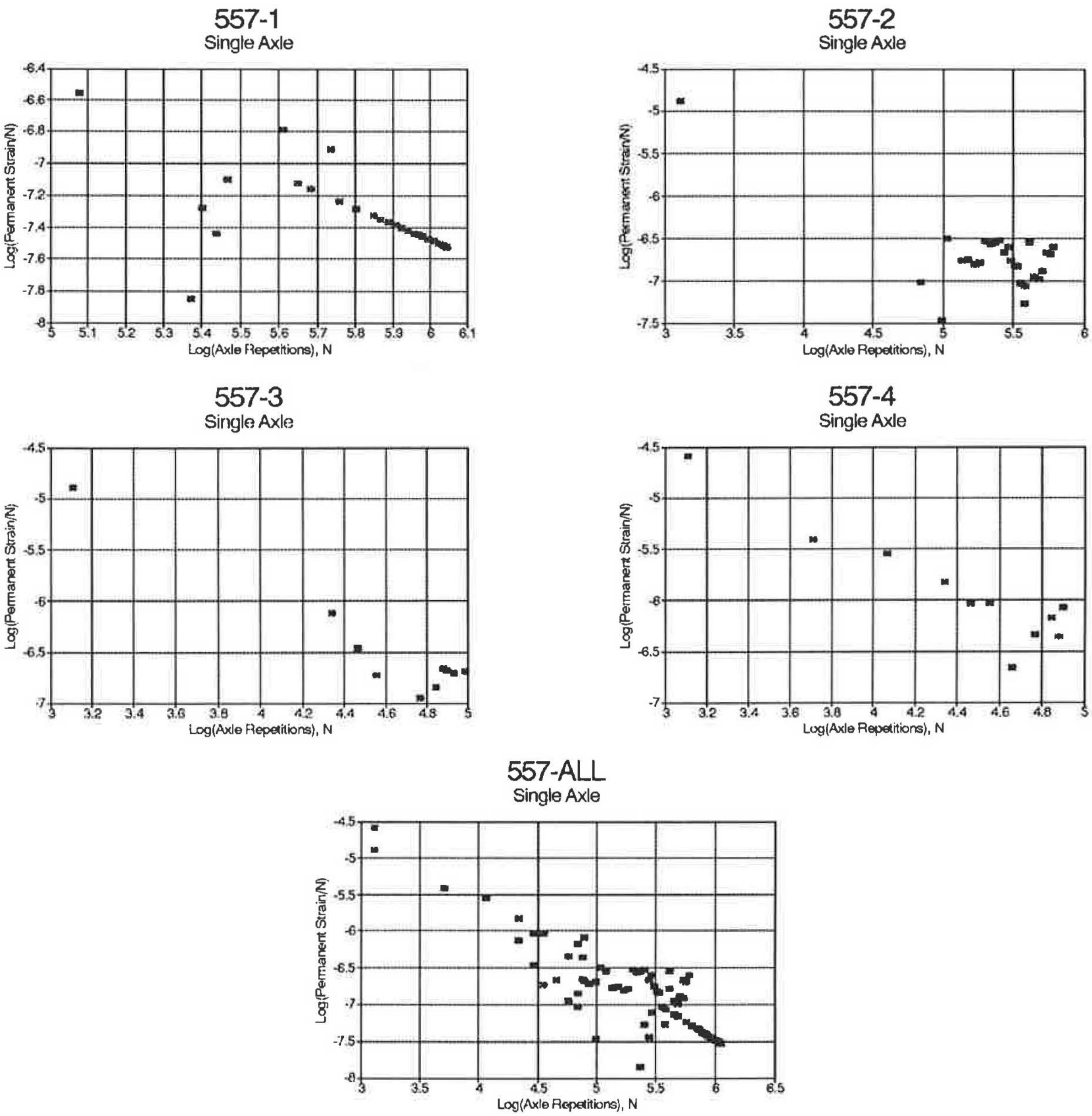


**FIGURE 1 Rut data for AASHO Road Test, Loop 4, Section 557, inner lane.**

dividing by the appropriate axle repetitions. These LEF values are given in Table 3.

A preliminary evaluation of the adequacy of these LEF values can be made by comparing them with the standard AASHTO LEF values. Figures 5 and 6 show the comparison of the rutting LEFs for the single-axle data for a flexible and a rigid pavement, respectively. This comparison indicates that single-axle rutting LEFs follow those for a very thick flexible pavement ($P_t$ = 2.5) or for a relatively thin concrete pavement quite precisely for the 0.5-in. rut depth failure criteria. If smaller rut depths were chosen as the failure level, the LEF values would be smaller, necessitating use of LEF charts for thinner concrete or thicker flexible pavements.

The comparison of the rutting LEF values for the tandem axles indicates quite a different performance picture. Figures 7 and 8 compare the tandem-axle rutting LEF values for rigid and flexible pavements, respectively. The relationship for the rigid pavement, thin slab, is again very close to that of the AASHTO LEFs. The notable exception is the heavy 48-kip tandem axle. Given the excellent correlation for the lighter tandem-axle and single-axle loads of similar magnitude and the fact that these similar loads did not cause the pavement to fail, the indication is that tandem-axle loads at these levels produce significantly higher damage due to rutting than other distresses such as roughness. The LEF comparison in Figure 8 for flexible pavement indicates that the flexible AASHTO LEF values are seriously in error when applied to rutting, with the tandem LEF values for flexible pavements being very low as compared with the values determined from the field data.

The AASHO Road Test data analyzed here indicate that the LEF for rutting accumulation in an asphalt concrete mix

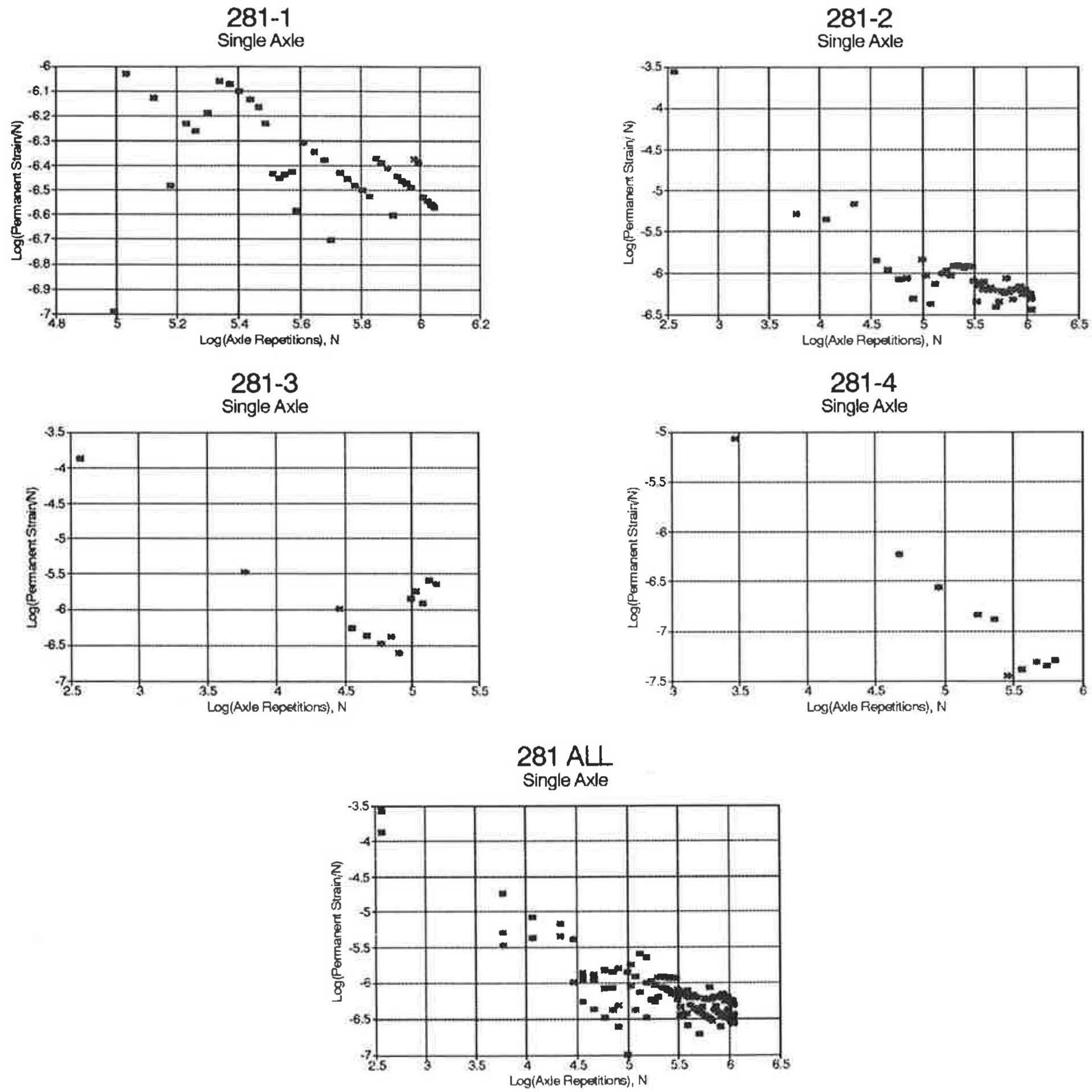


**FIGURE 2** Rut data for AASHO Road Test, Loop 6, Section 281, inner lane.

**TABLE 1** Regression Analysis of AASHO Road Test Rut Data

| LOOP | SECTION | SINGLE AXLE | | | TANDEM AXLE | | |
|---|---|---|---|---|---|---|---|
| | | A | B′ | $R^2$ | A | B′ | $R^2$ |
| 4 | 557 | -2.6801 | -.7800 | .59 | -2.4408 | -.7909 | .83 |
| 4 | 563 | -3.5947 | -.5908 | .42 | -3.0560 | -.6714 | .72 |
| 5 | 471 | -3.2364 | -.6792 | .55 | -3.1948 | -.6675 | .73 |
| 5 | 465 | -4.0637 | -.5098 | .47 | -3.7730 | -.5527 | .54 |
| 6 | 281 | -3.6365 | -.5732 | .69 | -2.7493 | -.6110 | .76 |
| 6 | 289 | -2.7244 | -.7607 | .62 | -2.8500 | -.7039 | .85 |

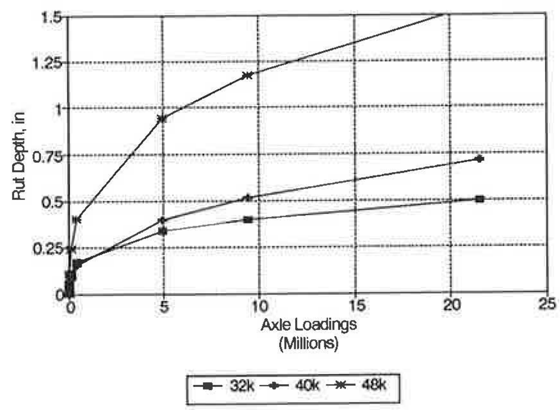


**FIGURE 3 Rut accumulation curves for tandem-axle sections.**

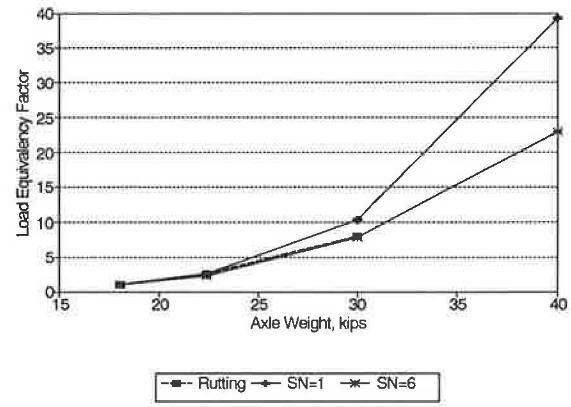


**FIGURE 5 Single-axle LEFs, flexible pavement.**

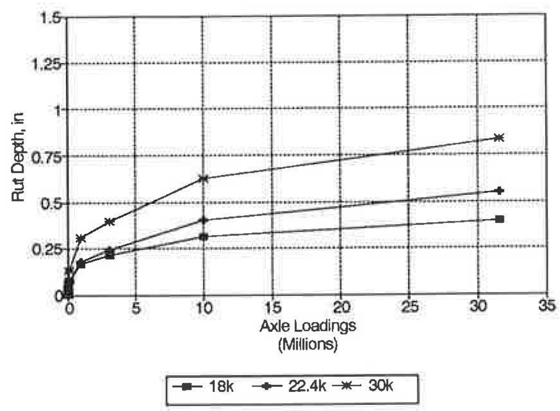


**FIGURE 4 Rut accumulation curves for single-axle sections.**

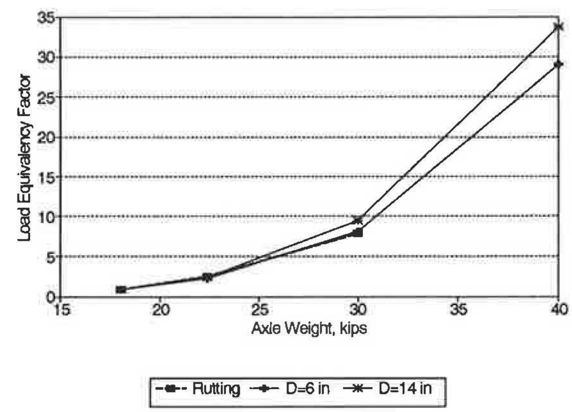


**FIGURE 6 Single-axle LEFs, rigid pavement.**

**TABLE 2 Load Repetitions to Produce Specified Rut Depths**

| | LOAD REPETITIONS TO INDICATED RUT DEPTH | | |
|---|---|---|---|
| AXLE TYPE | 0.3 in | 0.4 in | 0.5 in |
| 18 k single | 6,165,950 | 15,310,875 | 30,902,954 |
| 22 k | 3,548,134 | 7,079,458 | 12,105,981 |
| 30 k | 954,992 | 2,089,296 | 3,890,451 |
| 32 k tandem | 3,235,937 | 9,440,609 | 21,527,817 |
| 40 k | 2,371,374 | 4,931,738 | 8,709,636 |
| 48 k | 177,828 | 416,896 | 785,235 |

**TABLE 3 LEFs for Rutting From AASHO Road Test**

| | LEF FOR SPECIFIED RUT DEPTH | | |
|---|---|---|---|
| AXLE TYPE | 0.3 IN | 0.4 IN | 0.5 IN |
| 18 Kip single | 1.0 | 1.0 | 1.0 |
| 22.4 Kip | 1.74 | 2.16 | 2.55 |
| 30 Kip | 6.46 | 7.33 | 7.95 |
| 32 Kip Tandem | 1.91 | 1.62 | 1.44 |
| 40 Kip | 2.6 | 3.10 | 3.55 |
| 48 Kip | 34.67 | 36.7 | 19.09 |

most closely follows the level and trend established at the AASHO Road Test for the deterioration of a thin, rigid pavement. Only the single-axle LEF for rutting followed the trend of a very thick flexible pavement (SN = 6). This could be expected from a thick pavement that protects the base and subgrade from fatigue and develops little permanent deformation in the lower layers. For rutting development the tandem-axle effect on the asphalt concrete surface produces two somewhat distinct load pulses, and a tandem-axle LEF for rutting should be larger than an LEF for total damage—

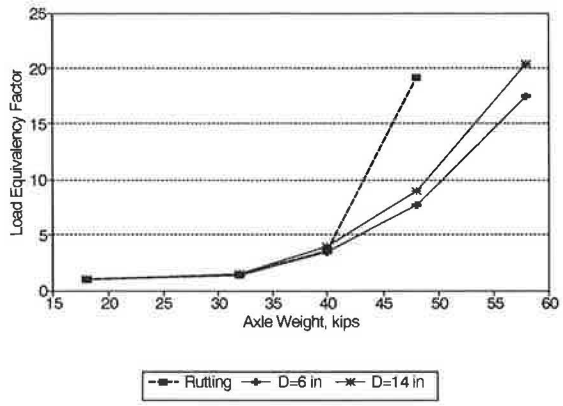


FIGURE 7 Rigid pavement LEFs, tandem axle.

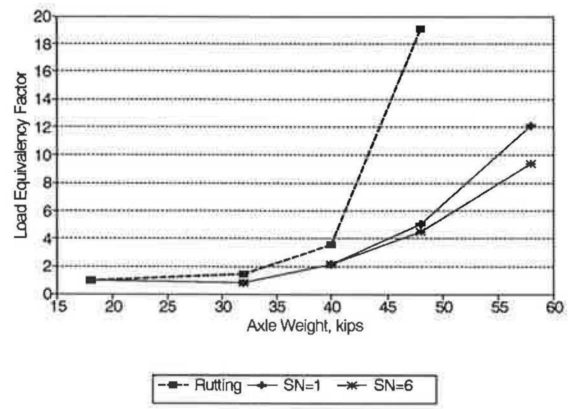


FIGURE 8 Flexible pavement LEFs, tandem axle.

much closer but not quite equal to twice the single-axle LEF value. This intuitive conclusion is substantiated by the field measurements analyzed and discussed here.

## Data Anomaly

The 48-kip tandem-axle data produced LEF values that were significantly different from the consistent relationships established by the other sets of data. This is most clearly shown in Figure 7, in which the mixture has entered the unstable portion of its permanent deformation life, the tertiary mode of failure. The mix subjected to single-axle loads, even at 30 kips, did not become unstable after 1 million axle repetitions. The tandem axle, however, produces twice as many wheel loads per axle pass. This doubling of the repetitions clearly caused the bituminous mix to fail, placing it in the unstable deformation mode. This is clearly evident in the data plotted as shown in either Figure 1 or 2. There is a distinct flattening of the data above 1 million wheel repetitions (500,000 axle repetitions), indicating unstable mix behavior.

## LEF Relationships

### *Single Axles*

The two different trends in the LEF data developed in this study can clearly be seen in equations developed to relate LEF values to terminal rut depth and axle load. The relationship for the single axles is

$$LEF = 1.83 \times 10^{-5} (RD)^{0.3854} (\text{SWeight})^{3.89}$$

where

LEF = load equivalency factor,
$RD$ = terminal rut depth selected for the criteria (in.), and
SWeight = axle weight (kips).

This relationship had an $R^2$ of 0.99 with a standard error of the estimate of 0.041 (on the log of the LEF). The exponent of the axle weight is very close to the standard exponent of 4 normally assigned to axle weight ratios on flexible pavements when PSI loss is examined. This relationship can be used to calculate LEF values for rutting comparisons with single axles of varying weights.

### *Tandem Axles*

The relationship for tandem-axle loadings, excluding the heavy 48-kip weight, which indicated a failed asphalt mixture, is

$$LEF = 1.113 \times 10^{-4} (RD)^{0.0279} (\text{TWeight})^{2.778}$$

where the variables are as defined before and TWeight is the tandem-axle weight. This relationship had an $R^2$ of 0.87 and a standard error of the estimate of 0.075 (on the log of the LEF).

The relationship for tandem axles does not provide the accuracy of the single-axle equation and probably should not be used for calculating LEF values for tandem axles because it includes a failed mix at the high-axle-load data point. The relationship and data clearly indicate the decreased impact of increased axle loads for tandem axles as compared with increased loads for single axles.

## FIELD COMPARISONS

As part of a layer coefficient study in Wisconsin (*7*), 31 flexible pavement sections were structurally evaluated. Fifteen of the 31 sections from which cores were taken were tested in the laboratory for permanent deformation. These pavement sections were relatively young, all less than 5 years of age. The 15 cores, representing virgin and recycled mixes, were carefully prepared for repetitive load testing. The $A$ and $B'$ parameters were determined on data taken from 5,000 load repetitions at a deviator stress level of 20 psi and a temperature of 70°F. These conditions roughly approximate the yearly conditions at the AASHO Road Test for an 18-kip single axle and allow a comparison to be made between the field mixes and the AASHO values. A more thorough examination would perform repeated load testing at several temperatures and stress levels, but such a testing program would not provide any better data for a better comparison with the AASHO mixture itself.

Using the $A$ and $B'$ coefficients calculated from the AASHO Road Test for the 18-kip loads in Loop 4, a probable range for these parameters can be established. The $A$ and $B'$ values for the 18-kip ESAL loading on the AASHO mixture are

- Low range: $A = 1.58 \times 10^{-3}$; $B' = -0.77$.
- High range: $A = 1.00 \times 10^{-3}$; $B' = -0.69$.

The rut depth accumulation curves for a 3-in.-thick mixture were calculated for the recycled and virgin mixes. Figure 9 shows several of the virgin Wisconsin mixes with the AASHO curves superimposed. The curves were generated using rutting Equation 2 with the appropriate $A$ and $B'$ coefficients from the laboratory testing. These data indicate that the rutting potential of the mixes tested is generally comparable with that of the AASHO mixture, with some extremes. Each mix was compared with the AASHO mixture in terms of general rutting potential. Table 4 shows the relationship between the modulus for the Wisconsin mixes tested and the rutting potential compared with those for the AASHO mixture. On average, the stiffer mixes (486,000 psi modulus) had better rutting resistance (low rating, rut curves below the AASHO curves), and the mixes with rutting potential similar to that of the AASHO mix (curves between the AASHO low or high curves) had a modulus in the range of 426,000 psi (at 70°F), very close to the AASHTO standard mixture modulus of 450,000 psi at 68°F. The mixes with high rutting potential compared with the AASHO mix (rut curves well above the high AASHO curve) had lower modulus values (312,500 psi). This relationship is shown in Figure 10 for the various levels of rutting potential. Although there is a high degree of variability, with stiff mixtures rutting more than softer mixtures, within this data set a general statement can be made for increased rutting in a mix with less stiffness. This general statement is not a hard and fast rule that ties modulus and rutting potential together, however, because the amount of scatter in the data is relatively high.

The AASHTO structural design procedure mandates that a lower structural layer coefficient be assigned to a mix with lower stiffness than that of the AASHTO standard mix. The Wisconsin data and the data from other laboratory investigations would indicate that a less stiff mix would generate more rutting than the stiffer mix. Thus, the structural layer coefficient of a mixture should not be reduced on the basis of stiffness alone because the potential exists that a less stiff mixture may have more rutting potential and deteriorate more quickly. A more appropriate design approach is to recognize that use of a mix with less stiffness or higher rutting potential, or both, will decrease pavement life compared with the AASHO mix because of the more rapid development of rutting. With the AASHO mixture parameters and the current mix parameters developed from laboratory tests, this loss of life can be calculated and used to adjust the layer coefficient, producing a coefficient more representative of a lower-quality mix that shortens pavement life because of increased rutting. This approach provides a means of assessing the impact of the mix on pavement life, but the "adjusted" layer coefficient must never be used in an original thickness design process. Unless specific comparison tests are made on new mixes, no new layer coefficient should be assigned without some indication of field performance.

**TABLE 4 Relationship Between Modulus and Rutting**

| Rutting Potential | Resilient Modulus, psi | Section ID |
|---|---|---|
| High | 363,000 | 2-4 |
| High | 262,000 | 4-2 |
| AASHO High | 663,000 | 1-1 |
| AASHO High | 156,000 | 7-1 |
| AASHO | 242,000 | 5-3 |
| AASHO Low | 513,000 | 2-1 |
| AASHO Low | 555,000 | 2-3 |
| AASHO Low | 312,000 | 2-6 |
| AASHO Low | 657,000 | 1-6 |
| Low | 500,000* | 2-5 |
| Low | 625,000 | 1-3 |
| Low | 100,000* | 7-2 |
| Low | 516,000 | 4-3 |
| Low | 478,000 | 4-1 |
| Low | 700,000* | 1-5 |

* Modulus estimated from indirect tensile strength relationship

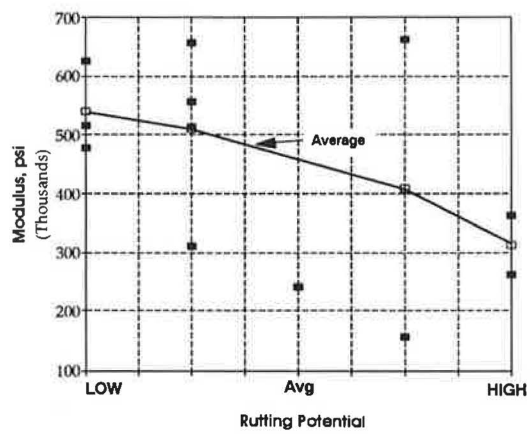


**FIGURE 10 Modulus versus AASHO mixture rutting potential.**

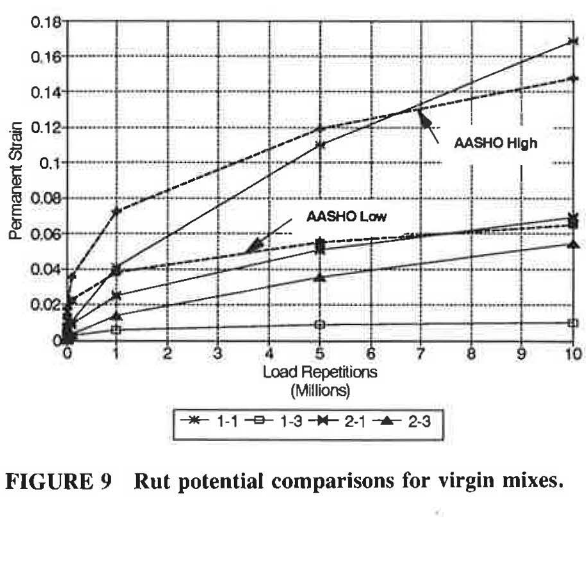


**FIGURE 9 Rut potential comparisons for virgin mixes.**

## SIGNIFICANCE OF RESULTS

When the progression of rutting is studied, the vehicles in a traffic stream must be converted to ESALs for design-life comparisons. Common methods of evaluating mix performance today involve measuring the rutting on a well-designed flexible pavement or on a rigid pavement that has been overlaid. To compare the performance of the mixes, the rut depths are typically presented as a function of the accumulated ESALs. If the LEFs used to obtain the ESALs are those of the underlying pavement, whether flexible or rigid, significant error can result, even if the mixes themselves are performing identically. In this case the difference arises from an improper LEF value for the tandem axles in the traffic stream.

The misapplication of the LEF value can present a problem when rutting performance is compared between different flexible pavements that are developing rutting only in the asphalt concrete layer. A hypothetical situation could exist in which two pavements have traffic streams that are very different but the same number of ESALs could be calculated. With different traffic streams the ESALs calculated for rutting comparisons using flexible AASHTO LEFs are not correct. An increased number of tandem axles in one of the traffic streams could actually cause more rutting in a shorter time because of the higher LEF that should be used compared with the LEF for flexible pavement. Use of the flexible pavement LEFs results in severe underprediction of the number of ESALs, which provides a distorted relationship of traffic level and rut depth development. The pavement with more tandem axles incorrectly appears to develop rutting at an accelerated rate. If the correct LEF values had been used, a more consistent relationship would have been developed.

The improper use of the tandem-axle flexible pavement LEF would represent approximately a 45 percent discrepancy in the estimation of the 18-kip ESALs required to rut an asphalt concrete mix to a depth of 0.5 in. Given that a traffic stream can have 25 percent trucks, which are typically equipped with tandem axles and contribute more than 80 percent of the total ESALs in the traffic stream, the underprediction of flexible pavement ESALs could amount to 30 percent, providing a false service life for mixture comparisons.

When new asphalt mixes are tested in the laboratory to develop rutting coefficients, they are tested under a standard stress level or series of stress levels. The $A$ and $B'$ coefficients can be used in the rutting equation to calculate rutting for an increased number of loadings for comparison with the AASHO mixture and for field comparison of this mix with others. These new mixes should do much better than the AASHO mix, which became unstable after very few heavy tandem-axle loads. Studies on rutting should be directed toward establishing this limiting number of load repetitions for unstable performance, as well as the mix parameters applicable to the stable deformation phase. All mixes will eventually reach the limit and initiate unstable performance. Current technology ignores this.

## ACKNOWLEDGMENTS

The data presented in this paper were developed as part of NCHRP Project 1-26 (*8*), conducted at the University of Illinois at Urbana-Champaign, and a report for the Wisconsin Department of Transportation conducted by ERES Consultants (*7*). The assistance of the authors of these studies is gratefully appreciated.

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# Selection of Subgrade Modulus for AASHTO Flexible Pavement Design

ROBERT P. ELLIOTT

The resilient modulus value (3,000 psi) used to represent the AASHO Road Test subgrade in the AASHTO flexible pavement design equation is examined. The subgrade modulus for pavements designed by the AASHTO *Guide for Design of Pavement Structures* must be consistent with this value for the guide to be used correctly. For cohesive soils, it is shown that this requires that laboratory values be based on tests conducted unconfined using a deviator stress of 6 psi. For backcalculated modulus values from cohesive soils to be consistent with the guide, the backcalculated modulus must be multiplied by a factor no greater than 0.33. Unmodified backcalculated values are unconservative. Laboratory values are also unconservative when the tests are conducted with a confining pressure and deviator stresses less than 6 psi. However, if the subgrade soils are noncohesive, it is not clear what the laboratory test conditions need to be or whether backcalculated $M_r$-values need to be adjusted to be consistent with the AASHO subgrade value. Appropriate values for noncohesive soils need further study.

The 1986 AASHTO *Guide for Design of Pavement Structures* (*1*) adopted the resilient modulus as the subgrade soil property controlling the design of flexible pavements. The introduction of the resilient modulus into routine pavement design caused considerable concern and discussion among engineers responsible for pavement design and materials testing. Most engineers routinely involved in these activities had little if any knowledge or understanding of the resilient modulus at the time that this variable was adopted, and practically none of the highway agencies had the capability to perform resilient modulus tests. Even today, the number of agencies equipped to perform resilient modulus tests on a routine basis are limited. Most agencies continue to perform the soil tests they used before the 1986 guide and rely on "correlations" between those tests and the resilient modulus to obtain the subgrade values needed in design.

Most of the discussions concerning the resilient modulus to date have centered on test methods, equipment, and repeatability. The concern has been so great, in fact, that AASHTO voted out the method of test that was adopted as a standard before the 1986 guide (AASHTO T274). As a result, there has been no standard method test for the resilient modulus even though the AASHTO guide uses it as a controlling design parameter. AASHTO recently balloted on the adoption of a revised method of test that may remedy this unusual situation.

However, there are other, more fundamental questions that need to be discussed, questions that probably were considered by the developers of the 1986 guide but that have not been conveyed adequately to the designers who bear the responsibility of using and interpreting the guide. These questions relate to the selection of the "correct" resilient modulus value to use in design. The "normal" practice of selecting a design modulus based on expected stress conditions under the pavement leads to the use of an unconservative value. Similarly, backcalculated resilient modulus values typically used for overlay design are unconservative unless reduced to be consistent with the modulus value used in the AASHTO design equation for the AASHO Road Test subgrade.

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## BASIC CONCEPT OF SUBGRADE RESILIENT MODULUS

The resilient modulus test is intended to examine the behavior of the soil as a support system for the pavement. In this respect the concept and basic approach to testing are very simple. When a heavy vehicle passes over the pavement, a dynamic stress pulse is transmitted to the soil. This stress causes the soil to deform, which in turn permits the pavement to deflect and bend. The stresses and strains generated within the pavement as a result of the deflection and bending are the factors that control the pavement performance. Thus, the pavement performance is directly influenced by the load-deformation behavior of the soil.

The basic concept of the resilient modulus test is to duplicate and measure this behavior in the laboratory. The test is normally conducted by placing the soil specimen in a triaxial cell and subjecting it to a confining pressure. The confining pressure is intended to simulate the confinement the soil would experience under the pavement. Dynamic load pulses are applied to the soil, and the resulting deformation or specimen strain is measured. The load pulse durations are typically about 0.1 sec and are intended to simulate the stress pulse in the subgrade caused by the passage of a heavy vehicle. The resilient modulus ($M_r$) is calculated from the load and deformation using the following equation:

$$M_r = \sigma_d / \varepsilon_r$$

where $\sigma_d$ is the stress caused by dynamic load pulse, also referred to as the deviator stress, and $\varepsilon_r$ is the resilient or recoverable strain.

Note that there are two components to the total specimen deformation, a resilient or recoverable portion and a permanent portion. Only the resilient portion is included in the resilient modulus. Except when the load applied is very close to the shear strength of the soil, the permanent deformation portion is very small and generally can only be measured as an accumulation of deformation over a large number of load repetitions.

To this point the concept, testing, and application of subgrade resilient modulus appear reasonably simple. It becomes quite complicated, however, when one discovers that there is no single resilient modulus for the soil, but rather an almost infinite number of values depending on test and sample conditions. For example, the resilient modulus of cohesive soils typically decreases quite significantly as the dynamic load or deviator stress increases. Conversely, higher confining pressures can result in some increase in resilient modulus. Specimen moisture content can have an overwhelming effect on the modulus value (which is lower with higher moisture contents). Other factors that can have an effect are density, freeze-thaw cycles, and method of compaction.

The selection of a design resilient modulus may still seem reasonably straightforward, because the objective of the test is to duplicate the soil's behavior as a pavement support system. It would appear, therefore, that the appropriate modulus would be the modulus determined for conditions consistent with the soil in its final location and condition under the pavement and at stress levels consistent with the stresses generated by a heavy vehicle load.

Ideally, this would be the case and should be the case for a fully developed mechanistic design procedure. However, determination of the modulus in the foregoing manner is not correct when the AASHTO guide procedure is used. In most cases, resilient modulus values selected on the basis of deviator stresses and confining pressures consistent with actual pavement conditions will be unconservative when used with the AASHTO guide. As a result, pavements designed using such values may be underdesigned or at least will have a lower level of reliability than the designer intends them to have. This is due to the empirical nature of the guide procedure.

## AASHO ROAD TEST TO AASHTO GUIDE

The AASHTO guide design procedure was developed as a modification of the AASHO Road Test performance equation (*2*). As such, all design inputs must be consistent with either Road Test conditions or conditions used in extending the performance equation beyond the Road Test. In particular, the subgrade resilient modulus value must be consistent with the modulus value used to represent the AASHO Road Test subgrade.

Soil type and condition were not variables in the AASHO Road Test. Every possible effort was made to ensure that the subgrade under all portions of the Road Test pavements was as uniform as possible. As a result, the performance equations developed from the Road Test did not include any measure of soil strength or condition. In adapting the equations for design, the earlier AASHTO guides simply used a "soil support scale" to represent changing subgrade conditions without fully defining the scale or what test should be used with it.

In the development of the 1986 guide, the soil support scale was abandoned and replaced with a relationship based on the resilient modulus. When this was done, a value of 3,000 psi was used to represent the AASHO Road Test subgrade. For the guide to be used correctly, the subgrade resilient modulus value used must be consistent with the 3,000 psi used for the AASHO subgrade.

The AASHTO guide and appendixes (*1*) do not indicate how or why this value was selected. However, a basis for the 3,000-psi value can be found in the literature. A value of 3,000 psi for the AASHO subgrade was suggested (perhaps for the first time) by Skok and Finn in a paper presented at the 1962 International Conference on the Structural Design of Asphalt Pavements (*3*). Skok and Finn derived this value from Benkelman beam deflection data. At the same conference, Seed et al. (*4*) presented laboratory resilient modulus data from tests on the AASHO subgrade soil (Figure 1). Their data show 3,000 psi to be a reasonable value if the deviator stress is greater than 12 psi when kneading compaction was used or 25 psi when static compaction was used. The confining pressure with their tests was 3.5 psi.

Thompson and Robnett (*5*) reported the most complete study of the resilient behavior of the AASHO subgrade. They performed detailed resilient modulus testing on a number of soils found in the state of Illinois. Their data from tests on the AASHO subgrade soil are summarized in Figure 2, from which it can be concluded that 3,000 psi is appropriate for the AASHO soil when it is about 1 percent wet of optimum and subjected to a deviator stress of about 6 psi or more. What is not apparent in Figure 2 but is discussed by Thompson and Robnett is that these values are based on tests without confining pressure. Their study found little effect due to confining pressure in the 3- to 5-psi range if the soil being tested was cohesive and compacted at or wet of optimum.

On the basis of the extensive testing of the AASHO soil by Thompson and Robnett, it may be concluded that the appropriate test conditions for subgrade resilient modulus when using the 1986 AASHTO guide are zero confining pressure and a 6-psi deviator stress. Nevertheless, the resilient modulus test method recently voted on by AASHTO calls for testing at a 3-psi confining pressure with deviator stress levels "selected to cover the expected in-service range." If the expected in-service range is less than 6 psi, the measured resilient modulus can be expected to be too high and unconservative (lower stresses generally result in higher resilient modulus values; see Figure 2).

For granular soils, the selection of appropriate test conditions is more complex and questionable. Obviously, noncohesive material cannot be tested in an unconfined state. Also the effects of stress level and confining pressure differ with granular materials. Figure 3 shows the typical stress state–resilient modulus behavior of a granular material. The bulk stress ($\theta$) referred to in Figure 3 is the summation of the deviator stress and three times the confining pressure (confining pressure in all three directions). Note that as the stress state increases, the resilient modulus also increases. How to consider this stress-dependent behavior within the empirical framework of the AASHTO guide so as to be consistent with the 3,000 psi used for the AASHO Road Test soil is not clear. Additional research on this point is needed. Thus, selection of the appropriate resilient modulus for a granular subgrade requires much thought and judgment from the designer.

## OVERLAY DESIGN AND BACKCALCULATED MODULI

The potential for selecting an unconservative resilient modulus value becomes even greater for the design of flexible

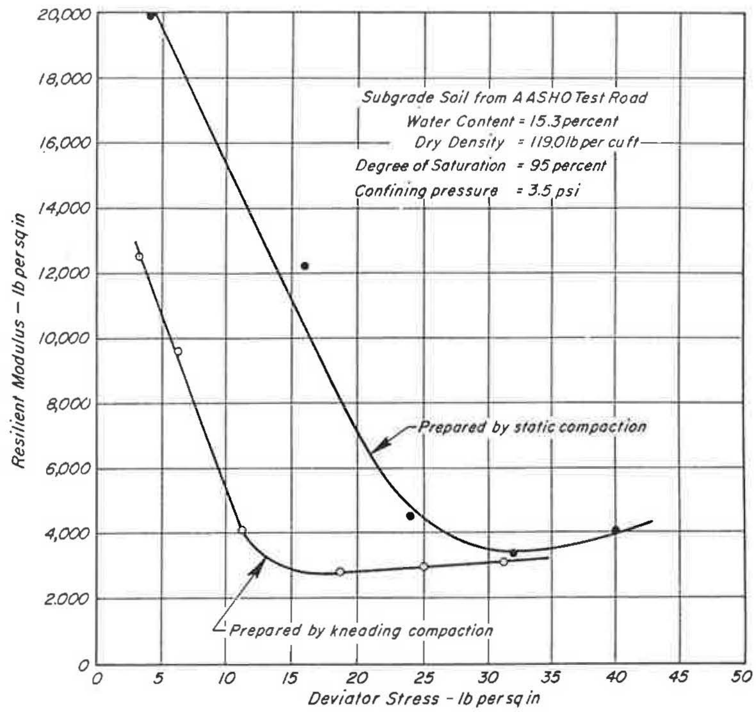


**FIGURE 1 AASHO Road Test subgrade resilient modulus tests reported by Seed et al. (*4*).**

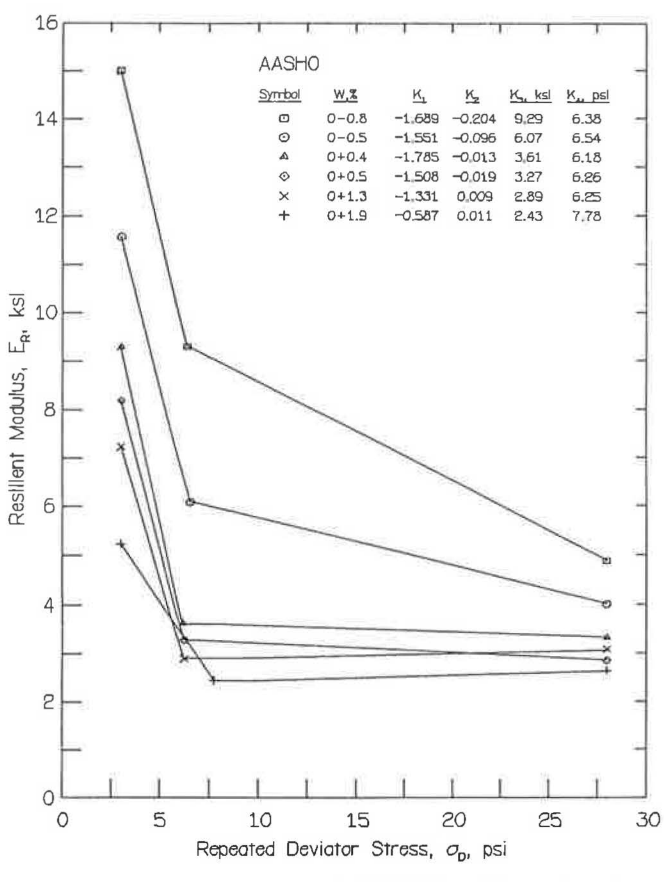


**FIGURE 2 Resilient modulus of AASHO Road Test subgrade reported by Thompson and Robnett (*3*).**

pavement overlays when nondestructive testing (NDT) and backcalculation are used (*6*). Most backcalculation schemes use the concept that load stresses are spread over a progressively greater area as depth into the pavement and subgrade increases (Figure 4). With this concept the surface deflection at some point sufficiently distant from the point of load application is due only to compression within the subgrade. The subgrade resilient modulus, therefore, is determined using only the surface deflection at this point.

An obvious problem with this concept when it is applied to the AASHTO guide is that the stress levels at this distant point are almost always much less than 6 psi. As a result of

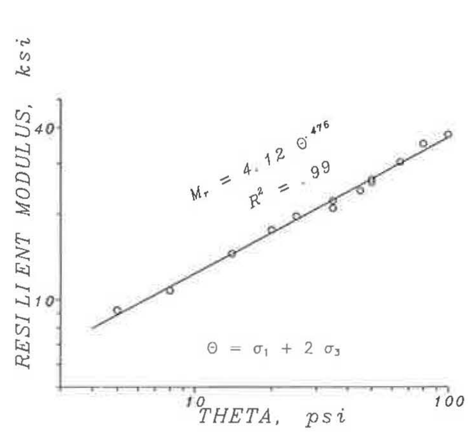


**FIGURE 3 Resilient modulus stress dependency typical of a noncohesive material.**

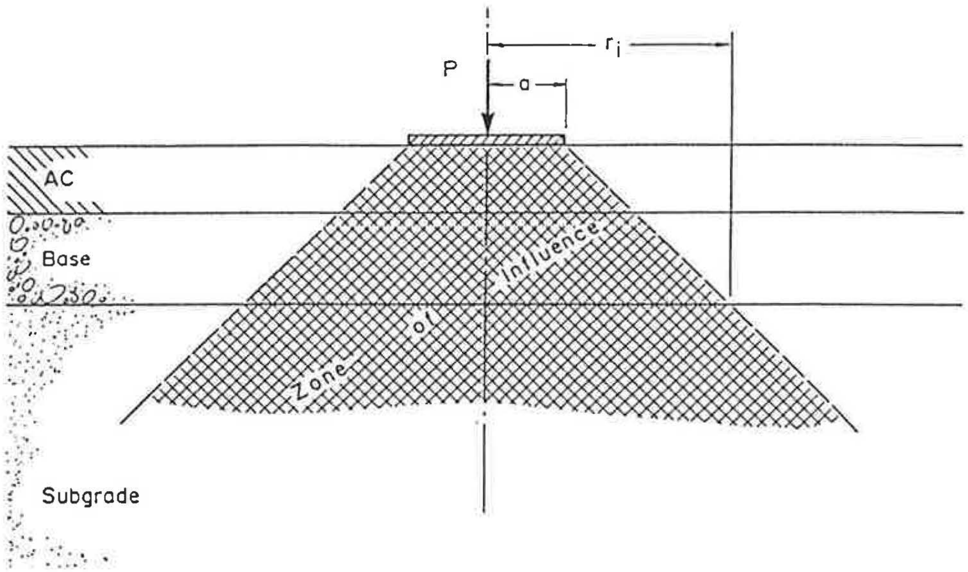


**FIGURE 4 Concept of the spreading of load stresses with depth below the pavement surface.**

the stress dependency of most soils, the backcalculated resilient modulus can almost always be expected to be too great to be consistent with the 3,000 psi used for the AASHO subgrade. Therefore, backcalculated resilient modulus values need to be adjusted when used with the 1986 AASHTO guide.

Unfortunately, NDT data suitable to be used to evaluate this backcalculation scheme are not available from the AASHO Road Test. However, Traylor (*7*) reported some NDT data that are reasonably suitable from measurements taken on Loop 1 of the Road Test several years after its conclusion. (Loop 1 pavements were not subjected to traffic during the Road Test and are still in place.) The NDT device was the FHWA Thumper used in the impulse load mode with a load magnitude of about 4,000 lb. In addition to the NDT tests, Traylor also reported laboratory resilient modulus results from Shelby tube samples taken shortly after the deflection measurements. The resilient modulus of these samples was measured using a deviator stress of 6 psi and no confining pressure. Therefore, the test results are consistent with the 3,000-psi value used in the AASHTO guide equation.

These data were used in an analysis directed at determining an appropriate adjustment factor for use of backcalculated resilient moduli with the AASHTO guide. The backcalculation was performed using the equation recommended for use in the AASHTO overlay design procedures (*6*):

$$M_r = 0.24P/d_r r$$

where $P$ is the applied load and $d_r$ is the deflection at a distance $r$ from the center of loading. Figure 5 is a plot of the backcalculated $M_r$-values versus the laboratory values from the Shelby tube samples. Except for one value, the laboratory results are reasonably consistent with the 3,000 psi used for the AASHO soil in the guide equation. However, the backcalculated moduli are greater than the laboratory values by a factor of 4.8 on average.

Two other analyses were performed to examine the need for the adjustment of backcalculated $M_r$-values. These analyses used the ILLI-PAVE finite-element program with the AASHO Road Test soil data reported by Thompson and Robnett. ILLI-PAVE models the stress dependency of cohesive soils as two intersecting lines, as shown by the Thompson and Robnett data. The inputs to the model are the slopes of the lines ($K_1$ and $K_2$), the point of intersection ($E_{ri}$ and $S_{di}$), and lower-limit deviator stress that sets a maximum limit on the resilient modulus. Using the data from Figure 2, the following values were selected to model the AASHO subgrade: $E_{ri}$ = 3,000 psi, $S_{di}$ = 6 psi, $K_1$ = 1.4 ksi/psi, $K_2$ = 0.01 ksi/psi, and lower-limit deviator stress = 2 psi (maximum possible $M_r$ = 8,600 psi).

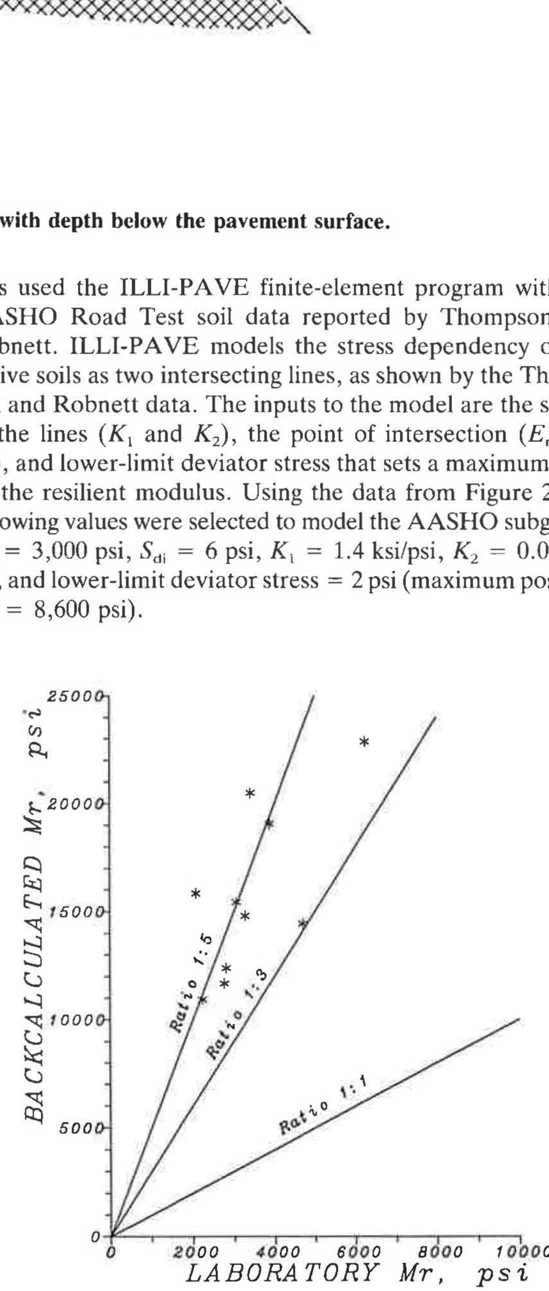


**FIGURE 5 Comparison of backcalculated values and laboratory resilient modulus values of AASHO Road Test subgrade.**

The first analysis was used to determine the $M_r$-value that ILLI-PAVE would assign for subgrade elements at radial distances from the center of loading and that would typically be used for subgrade backcalculation. The pavements analyzed had 3-in. and 5-in. asphalt concrete surfaces ($E_{ac}$ = 500 ksi) and aggregate bases ranging in thickness from 8 to 22 in. (base $M_r$ = 9,000 $\theta^{0.33}$). The loading was 9,000 lb on a 5.9-in. circular area [equivalent to a typical falling weight deflectometer (FWD) test]. Examination of the ILLI-PAVE output showed that at radial distances typically used for subgrade backcalculation, most elements would have the maximum possible $M_r$ (8,600 psi).

The second analysis used the deflection basins predicted by ILLI-PAVE. These deflections were used to predict subgrade $M_r$ using the backcalculation equation shown above. Deflections at distances ranging from 12 to 57 in. were used. The calculated modulus values ranged from 9,280 to 11,800 psi. There was no pattern showing the modulus increasing or decreasing with radial distance. Using the "best estimate" $M_r$ from each analysis, the mean backcalculated modulus was 9,806 psi.

These results suggest that if appropriate deflection data were available from the AASHO Road Test, the backcalculated subgrade resilient modulus would be greater than 3,000 psi by a factor of at least 3. Therefore, backcalculated values used with the AASHTO guide equation should be multiplied by a factor no greater than 0.33 to be consistent with the value assumed for the AASHO subgrade.

Similar results were obtained in a limited comparison study of FWD data and laboratory resilient modulus tests. Data were obtained from several projects located in three states (*6–8*) in which the FWD was used at a load of 9,000 lb. Subgrade samples from the deflection sites had been tested in the laboratory at a deviator stress of about 6 psi. Data from these tests are as follows:

| | *$M_r$ (psi)* | | *Ratio, Back-calculated to* |
|---|---|---|---|
| *Soil Type* | *Laboratory* | *Backcalculated* | *Laboratory* |
| A-2,A-4,A-6 | 7,000 | 25,000 | 3.6 |
| A-2,A-6 | 4,800 | 22,700 | 4.7 |
| A-4 | 3,000 | 27,500 | 9.2 |
| A-4,A-2-5 | 6,000 | 13,500 | 2.3 |
| A-7-6 | 6,000 | 19,600 | 3.3 |
| A-2-4 | 4,150 | 14,100 | 3.4 |
| A-4 | 4,500 | 14,300 | 3.2 |
| A-6 | 5,700 | 14,500 | 2.5 |
| A-4 | 7,650 | 13,400 | 1.8 |
| A-4 | 7,350 | 45,000 | 6.1 |
| A-6 | 6,000 | 45,000 | 7.5 |
| A-6 | 8,750 | 24,300 | 2.8 |

All of these analyses were for cohesive soils. The results apply primarily to stress-sensitive, fine-grained soils such as the AASHO subgrade. No attempt has been made to investigate granular, noncohesive soils. The need for adjustment of backcalculated values or for the appropriate laboratory test conditions for such soils is not clear. This subject needs investigation.

Designers need to be aware that the subgrade resilient modulus has a significant effect on the design structural number. They need to be very cautious in selecting the design resilient modulus. When unconservative values are used in design, pavements and overlays may be underdesigned or at least levels of reliability will be much less than those intended. Unexpected early pavement failures and excess maintenance costs may result.

## CONCLUSIONS

On the basis of the analyses presented in this paper, the following conclusions are drawn regarding the selection of an appropriate subgrade resilient modulus to use with the AASHTO guide method of flexible pavement design.

1. Laboratory resilient modulus tests on cohesive soils that are conducted at deviator stresses and confining pressures consistent with stress conditions expected below the completed pavement will result in $M_r$-values that are unconservative when used in the AASHTO guide.
2. For laboratory resilient modulus values on cohesive soils to be consistent with the value assumed for the AASHO Road Test subgrade, the test should be conducted unconfined using a deviator stress of 6 psi.
3. Subgrade resilient modulus values for cohesive soils backcalculated from NDT deflection data are unconservative when used directly with the AASHTO guide design equation.
4. Backcalculated $M_r$-values for cohesive soils need to be multiplied by a factor no greater than 0.33 to be consistent with the 3,000-psi $M_r$-value assumed for the AASHO Road Test subgrade.
5. If subgrade soils are noncohesive, it is not clear what the laboratory test conditions need to be or whether backcalculated $M_r$-values need to be adjusted. These points should be studied.

## ACKNOWLEDGMENT

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# Porous Pavement for Control of Highway Runoff in Arizona: Performance to Date

MUSTAQUE HOSSAIN, LARRY A. SCOFIELD, AND W. R. MEIER, JR.

In 1986 the Arizona Department of Transportation constructed a 3,500-ft-long porous pavement experimental test section on State Route 87 in the Phoenix metropolitan area. The objectives of the project were to determine the constructibility and subsequent performance of porous pavement as a drainage system and pavement structure in an urban area and a desert environment. The porous pavement test section has performed satisfactorily for 5 years. Although a slight decrease in the infiltration rate has occurred, both the infiltration rate and the storage capacity are above the design values. The storage capacity of the pavement subbase and trench drain system has been underutilized. If a design intensity storm occurs during the remaining service life, this should be verified. Visual observation during rain storms has shown that the surface of the porous pavement section does not include sheet flow, which provides a marked difference in stripe delineation and pavement glare during nighttime inclement weather driving as compared with conventional pavement. Mu-meter skid test results for the porous pavement section are comparable with those of conventional pavements (control). Material tests conducted on the pavement components indicate that the Marshall stability, resilient modulus, and asphalt cement viscosity of the open-graded asphalt concrete have increased with time. No cracking or significant surface deformation has occurred during the 5 years of service. Annual falling weight deflectometer testing was conducted to establish the changes in layer properties. To date, little change has occurred in the layer moduli except for the open-graded subbase, whose modulus has decreased with time. No unusual presence of moisture was detected in any layer of the pavement system. The subgrade moisture content has achieved equilibrium and less than optimum moisture content determined during the design process.

Paved surfaces increase runoff and overload the existing sewer systems if alternative drainage is not provided. Rainfall is the only source of surface runoff in the Phoenix area. Typical summer storms have high intensity and short duration, whereas typical winter storms have low intensity but longer duration (*1,2*). This creates a large volume of runoff requiring costly highway drainage systems. Up to 35 percent of the total cost of highway construction projects in Arizona's urban area is expended on drainage structures (*3*). In an attempt to reduce the need for extensive drainage systems, porous pavements have been suggested as an alternative to conventional pavement (*4,5*). The basic concept of porous pavement design is that in addition to carrying traffic, the porous pavement will also serve as a drainage system by absorbing and storing storm waters and dissipating them into the ground. In 1986 the Arizona Department of Transportation (ADOT) constructed a 3500-ft-long porous pavement experimental test section on an urban highway. The objectives of the project were to determine the constructibility and subsequent performance of porous pavement as a drainage system and pavement structure in an urban area and a desert environment.

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## PROJECT LOCATION AND LAYOUT

The test section is located in the three northbound lanes of State Route (SR) 87 (Arizona Avenue) between Station 105 + 00 and 140 + 00 in the city of Chandler between Elliot and Warner roads. Chandler is a rapidly growing and developing suburban city approximately 20 mi southeast of Phoenix. SR 87 is heavily traveled by commuter traffic going to and from the Superstition Freeway, which is approximately 2.5 mi north of the project. Currently, the average daily traffic is approximately 30,000. Figure 1 shows the layout of the porous pavement section and the control section.

## DESIGN CONSIDERATIONS

### Porous Pavement Section

The porous pavement section consists of 6 in. of open-graded hot-mix asphalt concrete, 6 in. of open-graded asphalt-treated base (ATB), and 8 in. of open-graded subbase. The pavement structure was designed equivalent to the control section of conventional dense-graded pavement. This pavement was designed using the AASHTO (*6*) design equation to carry the design traffic loading of 2,270,653 single-axle, equivalent 18-kip loads for a 20-year design period (*7*).

A woven filter fabric was placed for separation of the subbase and subgrade. The open-graded layers of the pavement drain into a trench at the edge of the pavement, which is filled with open-graded aggregate. The water from the drainage trench was expected to dissipate into the ground. An alternative drainage system was also provided for the experimental section as a backup in the event of failure of the designed experimental drainage system. The pavement structure designed was found to have adequate water-holding capacity to

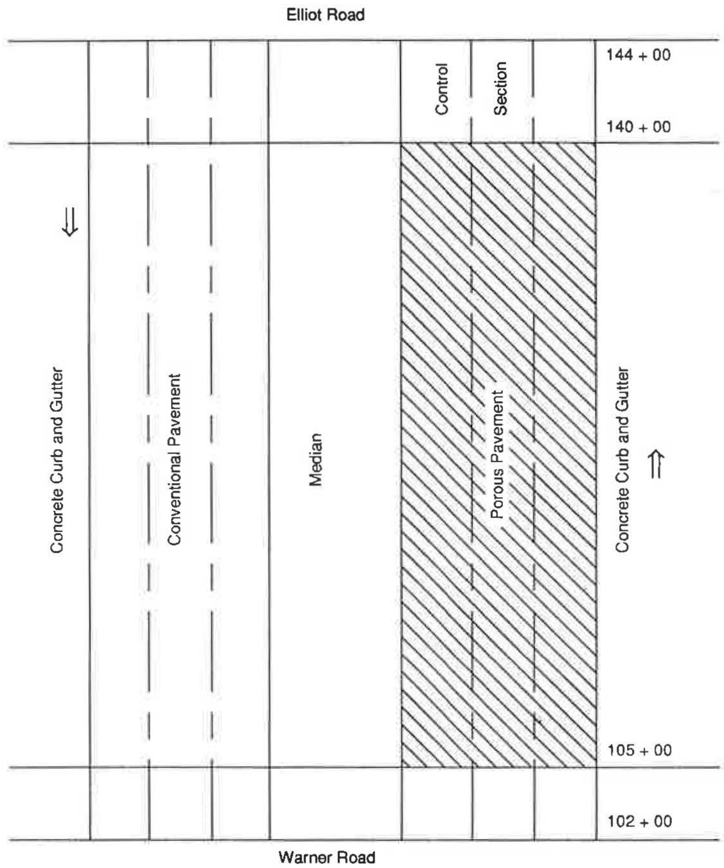


**FIGURE 1 Layout of porous pavement test section.**

temporarily store runoff from the 10-year 24-hr design storm. The intensity of this rain storm was found to be 0.11 in./hr. The design rainfall intensity for percolation of water through the surface course was based upon the 10-year frequency, 10-min duration storm. This intensity was estimated at 5.20 in./hr during the 10-min storm (*8*).

### Control Pavement Section

The control section of the project is located at the north end of the porous pavement section and consists of 8 in. of asphalt concrete over 7 in. of aggregate base. The same design was used for the three southbound lanes, which were used as a control section for Mays roughness and mu-meter testing. Figure 2 shows the structural section of the porous pavement and the control section.

The specific design data are summarized in Table 1. A design structural number (SN) of 4.5 was selected for both sections. The thicknesses of different layers were calculated on the basis of this structural number. A structural layer coefficient of 0.40 was assumed for the open-graded asphalt concrete.

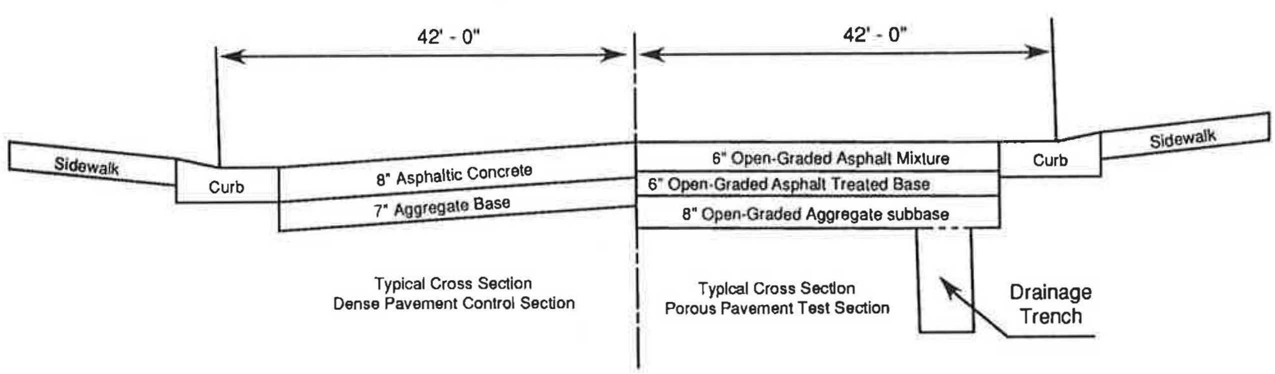


**FIGURE 2 Typical cross section of ADOT's experimental porous pavement.**

TABLE 1 Summary of Structural Design Data

| Material Type | Thickness (in) | Layer Coefficient | Structural Number |
|---|---|---|---|
| **CONTROL SECTION:** | | | |
| AC (1/2") (Control) | 2 | 0.44 | 0.88 |
| AC (3/4") (Control) | 6 | 0.44 | 2.64 |
| AB (Control) | 7 | 0.14 | 0.98 |
| | | | 4.50 |
| **POROUS PAVEMENT:** | | | |
| AC (Open Graded) | 6 | 0.40 | 2.40 |
| ATB (Open Graded) | 6 | 0.20 | 1.20 |
| Subbase (Open Graded) | 8 | 0.11 | 0.88 |
| | | | 4.48 |

**Other data: Design R-value = 15, 20-year 18K ESALS = 2,270,653, Regional Factor = 1.0, Terminal Serviceability = 2.5.**

### Open-Graded Asphalt Concrete Mix Design

The mix design for the open-graded asphalt concrete was based on Arizona Method 814a: Design of Asphaltic Concrete Friction Course (*9*). The method is primarily used by ADOT for designing open-graded wearing courses on dense-graded pavements. This method determines the bitumen content and density of an asphaltic concrete course. The design bitumen content for the porous asphalt concrete was 5.5 percent by weight of the total mix. The bitumen content of the ATB layer was 1.8 percent.

### Drainage Characteristics of Different Layer Materials

Samples of the pavement layer materials were tested in the laboratory for void content and coefficient of permeability. The coefficient of permeability of the asphalt concrete, ATB, and open-graded subbase were 200, 16,000, and 23,000 ft/day, respectively. The coefficients of permeability were used to estimate the rate at which the porous pavement could drain water from the surface. The air voids allowed a measurement of the water-holding capacity of the materials used (*10*).

### Drainage Trench Design

A parallel drainage trench was designed along the edge of the roadway and connected to the open-graded subbase such that runoff entering the pavement could readily flow into the trench. The trench was designed to drain the 10-year, 24-hr design storm from the subbase within 26 hr. A trench 2 ft wide by 4 ft deep was found to be adequate to drain one-half of the road plus the sidewalk and shoulder. The trenches were designed to be filled with the open-graded aggregate specified for the pavement subbase. The aggregate in the drainage trench and the subbase were designed to be isolated from the subgrade soil by a permeable geotextile at the soil-aggregate interface (*10*).

## CONSTRUCTION PROBLEMS

There were minor difficulties in the construction of the open-graded subbase and base. The major problem occurred when traffic was being carried on the east half of the open-graded asphalt concrete while the west half was being constructed. Within 3 weeks, severe vertical deformation of the pavement was noted. Surface deformations were measured with a straightedge placed on the pavement. Vertical displacements from the straightedge varied from ⅛ in. to 1 in. The average depth was ⅜ in., with 139 of the 144 readings taken being ⅝ in. or less. After investigation, it was concluded that as the hauling units deposited ATB material, decompaction of the untreated subbase occurred, resulting in an increased volume of the open-graded subbase. The volume of this subbase was decreased by recompaction of this course during subsequent construction and traffic. This change in volume of the untreated subbase is the most probable cause of the vertical deformation. The entire section was subsequently rolled with a steel vibratory roller for three or four coverages, which produced some leveling of the surface. An additional thin lift of open-graded asphalt was placed to produce a surface at the proper elevation and cross-slope. The project was finally opened to traffic in July 1986 (*10*).

## INSTRUMENTATION

### Rain Gauge

In order to obtain rainfall data, a continuously recording rain gauge was placed just beyond the west right-of-way line at Station 139 + 10. In August 1987, at the request of the property owner, the rain gauge was moved 385 ft to the west. Continuous rainfall readings were taken from June 1986 to July 1989.

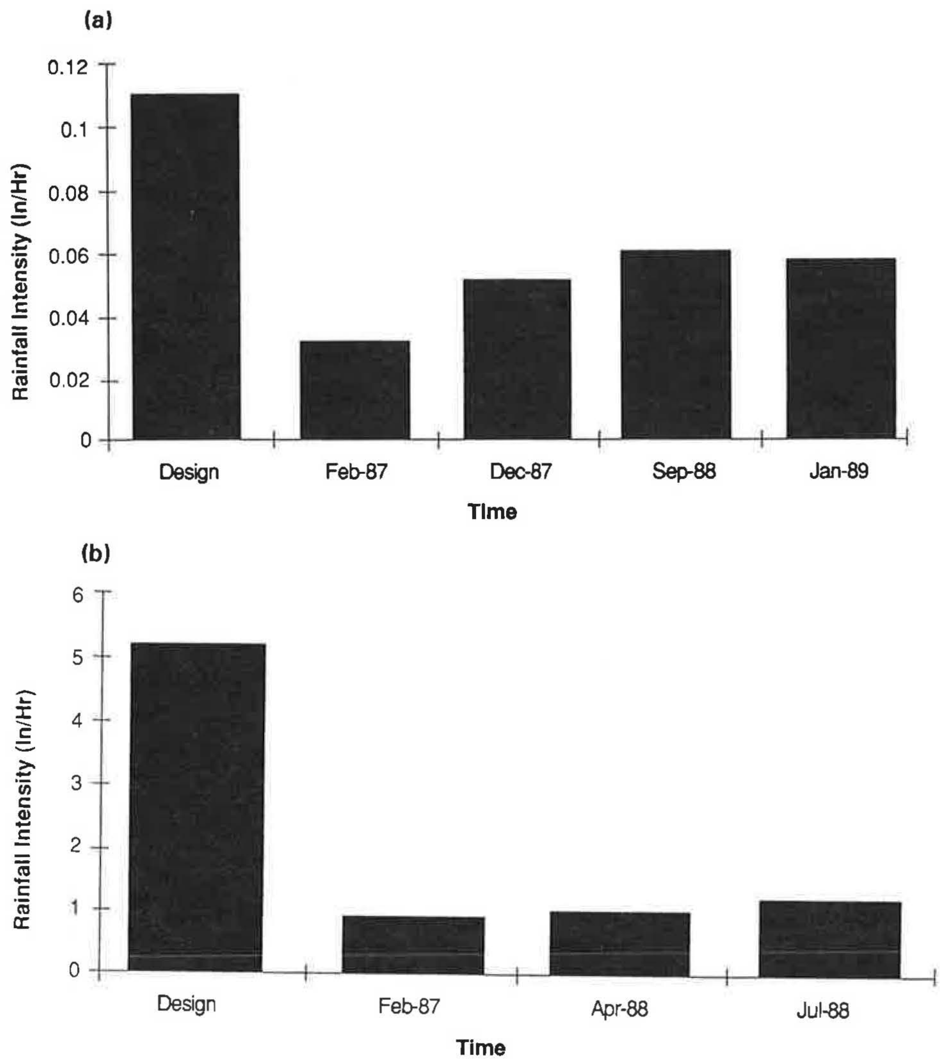


**FIGURE 3 Comparison of design and actual rainfall: (*a*) 10-year, 24-hr storm; (*b*) 10-year, 10-min storm.**

### Moisture Monitors

Soil moisture-monitoring devices (Soiltest soil moisture-temperature meter set, MC-312) were placed in the subgrade at two locations within the porous pavement and three locations in the control pavement. Six positions were monitored at each location. Moisture-monitoring locations were at Stations 97 + 40, 138 + 00, and 143 + 25 in the southbound lanes and Stations 108 + 00 and 138 + 00 in the northbound lanes. Cells were placed at depths of 1 and 3 ft below the top of the subgrade at distances of approximately 5, 10, and 20 ft from the front face of the curb and gutter.

### Well Point

A well point was placed within the drainage trench located in the east concrete gutter at Station 130 + 00. A device installed in the well point in February 1987 recorded the highest water level reached in the trench.

## FIELD INVESTIGATION SINCE CONSTRUCTION

### Visual Pavement Distress Surveys

The porous pavement section was visually reviewed several times between July 1986 to April 1990. These reviews included observing both the conventional and the porous pavement sections for cracking, distortion, disintegration, and frictional characteristics. However, no such distresses were observed during these reviews. Part of the outermost lane was inadvertently flushed in summer 1989 by maintenance personnel flushing an adjoining project.

### Rut Depth Measurements

Rut depth measurements were taken on all three lanes in 1986, 1987, and 1988 and on the outermost lane in 1989 and 1990. Currently, the average rut depth on the porous pavement section (0.20 in.) is higher than that on the conventional pavement (0.10 in.). However, the average rut depth on the porous pavement section is within an acceptable limit.

## NONDESTRUCTIVE TESTING

### Purpose and Background

To detect the structural deterioration of porous pavements as well as dense-graded pavements through the change in backcalculated moduli of the layers, nondestructive testing (NDT) has been conducted every year since 1988 using a falling weight deflectometer (FWD), Dynatest Model 8002, with sensor locations of 0, 12, 24, 36, 48, 60, and 72 in. from the center load at load levels of 6,000, 9,000, and 12,000 lb. The testing was performed in early fall at 15 locations in 1988, 27 locations in 1989, and 90 and 21 locations in 1991. These tests were performed on the outer wheelpath of the outside lane, generally 2 to 3 ft from the edge of the pavement. The FWD deflection data were analyzed using the BKCHEVM elastic layer computer program to backcalculate the layer moduli (*11*). Details of the analysis methodology may be found elsewhere (*12*).

### Backcalculation Results

The results of the backcalculation analysis are summarized in Table 2 for porous and conventional dense-graded pavements. The average open-graded asphalt concrete moduli typically ranged between 374 and 450 ksi. Individual test results ranged from a low of 205 ksi to a high of 751 ksi. The average asphalt

**TABLE 2 Backcalculated Layer Moduli Summary Statistics**

(A) POROUS PAVEMENT

| | | Backcalculated Layer Moduli (ksi) | | |
|---|---|---|---|---|
| **Layer** | **Year** | **Mean** | **Std. Dev.** | **Coeff. of Var. (%)** |
| Asphalt* Concrete | 1988 | 726 | 244 | 39 |
| | 1989 | 688 | 433 | 63 |
| | 1990 | 1110 | 414 | 37 |
| | 1991 | 527 | 221 | 42 |
| Asphalt Treated Base | 1988 | 302 | 180 | 60 |
| | 1989 | 624 | 239 | 38 |
| | 1990 | 589 | 227 | 39 |
| | 1991 | 232 | 175 | 76 |
| Subbase | 1988 | 20 | 19 | 95 |
| | 1989 | 15 | 19 | 130 |
| | 1990 | 13 | 18 | 138 |
| | 1991 | 9 | 6 | 67 |
| Subgrade | 1988 | 21 | 4 | 19 |
| | 1989 | 20 | 3 | 15 |
| | 1990 | 19 | 3 | 16 |
| | 1991 | 19 | 2 | 11 |

(B) CONVENTIONAL PAVEMENT

| | | Backcalculated Layer Moduli (ksi) | | |
|---|---|---|---|---|
| **Layer** | **Year** | **Mean** | **Std. Dev.** | **Coeff. of Var. (%)** |
| Asphalt* Concrete | 1988 | 1245 | 15 | 1 |
| | 1989 | 2163 | 328 | 15 |
| | 1990 | 2556 | 30 | 1 |
| | 1991 | 780 | 145 | 19 |
| Asphalt Treated Base | 1988 | 71 | 26 | 37 |
| | 1989 | 46 | 1 | 2 |
| | 1990 | 30 | 8 | 27 |
| | 1991 | 23 | 18 | 78 |
| Subgrade | 1988 | 27 | 3 | 11 |
| | 1989 | 26 | 1 | 3 |
| | 1990 | 24 | 1 | 6 |
| | 1991 | 23 | 1 | 3 |

* Temperature adjusted at 77°F.

concrete modulus for the dense-graded pavement is almost two times that for open-graded asphalt concrete. The subbase modulus for porous pavement and aggregate base modulus for dense-graded pavement tend to decrease with time. No suitable explanation for this trend can be offered at this time. Although there is fluctuation of asphalt concrete moduli for both types of pavement, this variation may be a function of the backcalculation procedure. The subgrade modulus values for both types of pavement are relatively constant over the period of time studied.

## PAVEMENT PERFORMANCE SINCE CONSTRUCTION

### Function Performance

*Roughness Testing*

Mays meter runs were made on all three lanes of porous pavement immediately after construction. The runs were repeated in 1989 in two consecutive months. At that time the three southbound lanes of the conventional pavements were tested for comparison. In April 1991 the measurements were repeated on the outer lanes of the porous pavement and the conventional pavement in the southbound direction. Table 3 shows the Mays roughness in inches per mile. The as-built roughness of the porous pavement is somewhat higher. The 3-year and 5-year roughness measurements show that the roughness values for the porous pavement are higher than those for the conventional pavement. Neither type of pavement has shown significant increase in roughness with time and use.

*Skid Testing*

Skid measurements are not available for this project immediately after construction. The measurements were taken on all three lanes of the porous pavement in April 1988 and repeated in December 1989 and February 1990. The later measurements also included all three lanes of the conventional pavement in the southbound direction. A mu-meter was used for the skid testing. Table 4 shows the mu-meter values resulting from each test. The 1990 measurements show a slight decrease of skid resistance as expressed by mu-meter values compared with the 1988 measurements, but the mu-meter values on the porous pavement and conventional pavement are comparable. The 1991 measurements show that mu-meter values on both sections are comparable.

**TABLE 3 Mays Roughness for Porous and Conventional Pavements**

| | Mays Roughness (ins/mile) | | | |
|---|---|---|---|---|
| Month/Year | Lane 1 | Lane 2 | Lane 3 | Average |
| **POROUS PAVEMENT:** | | | | |
| August, 1986 | 161 | 147 | 199 | 169 |
| June, 1989 | 166 | 140 | 215 | 174 |
| July, 1989 | 165 | 135 | 205 | 168 |
| May, 1991 | 174 | 81 | 148 | 135 |
| **CONVENTIONAL PAVEMENT:** | | | | |
| June, 1989 | 166 | 119 | 148 | 145 |
| July, 1989 | 175 | 116 | 147 | 146 |
| May, 1991 | 84 | 53 | 163 | 100 |

**TABLE 4 Mu-Meter Values for Porous and Conventional Pavements**

| | Mu-meter Values | | | |
|---|---|---|---|---|
| Month/Year | Lane 1 | Lane 2 | Lane 3 | Average |
| **POROUS PAVEMENT:** | | | | |
| April, 1988 | 53 | 51 | 53 | 52 |
| December, 1989 | 41 | 42 | 45 | 43 |
| February, 1990 | 49 | 44 | 46 | 43 |
| May, 1991 | 52 | 52 | 54 | 53 |
| **CONVENTIONAL PAVEMENT:** | | | | |
| December, 1989 | 46 | 49 | 44 | 46 |
| February, 1990 | 50 | 52 | 45 | 49 |
| May, 1991 | -- | 49 | 52 | 51 |

### Hydraulic Performance

#### *Analysis of Rainfall Data*

The rain gauge data collected by the continuous recording rain gauge were analyzed over a 3-year period from 1987 to 1989. During 1986–1987, the highest amount of rainfall recorded over a 24-hr period was 0.82 in. on February 24, 1987. The total rainfall for this period was 7.07 in. The greatest estimated rainfall intensity occurred on February 26, 1987, when 0.42 in. of rain fell in an estimated 0.5 hr, giving an estimated rainfall intensity of 0.84 in./hr. In 1987–1988, the highest amount of rainfall (1.15 in.) over a 24-hr period occurred on December 17, 1987. The greatest measured rainfall intensity (1.0 in./hr) occurred on April 15, 1988. The design rainfall intensities for a 10-year, 10-min storm and a 10-year, 24-hr storm are 5.20 and 0.11 in./hr, respectively. The rainfall intensity in April 1988 is one-fifth the amount of a 10-year, 10-min storm. The rainfall intensity in December was slightly less than half that for a 10-year, 24-hr storm.

In 1988–1989, the highest amount of rainfall (1.35 in.) over a 24-hr period occurred in September 22, 1988. The greatest measured rainfall intensity was 1.23 in./hr on July 29, 1988. This intensity is again much lower than that of the design 10-year, 10-min storm. The last highest amount of rain was recorded on January 4, 1989. A total of 1.24 in. was recorded over a 24-hr period.

Design rainfall intensities are compared in Figure 3 for a 10-year, 24-hr storm and a 10-year, 10-min storm with actual rainfall on the project in years 1987–1989.

The surface of both pavements was observed after rain storms in October 1986 and February 1987. It was visually observed that the surface of the porous pavement, although wet, did not have any standing or excess water on its surface. The surface of the conventional pavement was also wet, but sheets of water could be seen on the surface along with water flowing in the curbs. It appeared that the porous pavement was draining water faster than the conventional pavement. Also the lane stripe delineation was more visible on porous pavement than on conventional pavement. During night driving, reduced headlight glare was also observed.

#### *Analysis of Well Point Data*

The highest water level recorded with respect to the bottom of the trench during the first and the second years was 3½ and 9 in., respectively. The readings indicated that the capacity of drainage trench had not been exceeded. No recording is available for the third year.

#### *Subgrade Moisture Content*

All moisture monitors were read on June 7, 1986. The majority of the monitors then underwent a significant decrease in resistance before the next reading on August 11, 1986. A substantial portion of this change appeared to be stabilization of the monitors or moisture contents within the subgrade in the vicinity of the gauges. Readings were taken several times during the first year. The moisture readings from monitoring devices located at Station 97 + 40 Lt and 138 + 00 Lt remained approximately the same when read in August 1986, July 1987, and June 1988. After the first year, the estimated moisture content of the subgrade at Station 108 Rt increased approximately 2.3 percent. After 2 years, Station 108 Rt had an estimated increase in moisture content of 1.5 percent. The moisture content in the subgrade increased more for Gauges 5 and 6, which are located nearest the trench excavation.

The moisture content of the subgrade had decreased approximately 1 percent at Station 138 + 00 Rt. in 1988. This change was significant at Gauge 5, where the decrease was estimated to be 4.7 percent. The change in the moisture content at Gauge 5 may be caused by the stabilization of the subgrade moisture at this location.

Moisture monitor gauges located at Station 143 + 25 Lt indicated a fairly insignificant change in subgrade moisture except for Gauge 5. During the first monitoring period, Gauge 5 indicated a subgrade moisture increase of 9.2 percent. After 2 years of monitoring, the increase was 8.7 percent. This small change in moisture content indicates that the moisture content in this area of the subgrade has stabilized.

At all stations except 138 + 00 Rt, some gauges read 2000 or infinity. This indicates that the soil has dried out or that the monitoring gauge has malfunctioned (7).

### Structural Performance

No cracking has been observed in the porous pavement to date. Some raveling was suspected in 1988, but it did not turn out to be serious. The rutting in the porous pavement, though higher than in conventional pavement, is within acceptable limits.

## SAMPLING AND TESTING OF POROUS PAVEMENT MATERIALS

In 1990 an extensive sampling and laboratory testing of porous pavement materials was conducted. The objective of this testing program was to determine characteristics of in-service porous pavement materials. The test results were planned to be correlated with the functional and structural performance of the porous pavement. The test results were also to serve as a datum for future comparison of material properties.

### Sampling

The number of samples required to characterize the porous pavement materials was based on the statistical analysis of as-built porous pavement core properties. Thirty open-graded asphalt concrete cores were retrieved by dry coring. Nuclear density gauge readings were obtained at each coring location to measure the in situ density of porous asphalt concrete. The ATB material samples were collected at each location. Subbase samples were collected at 12 locations. The subgrade was sampled at six locations.

## TESTING AND RESULTS

### Bulk Density

Thirty cores were prepared for bulk density testing by saw cutting at the lift line between the asphalt concrete and the ATB layers. Bulk density tests were performed by measuring the length, diameter, and mass of each core as described in Procedure 6.2 of ASTM D3203. As mentioned earlier, nuclear density gauge measurements were taken at the core locations. Table 5 shows the summary statistics of the density measurements of the cores as well as the nuclear density gauge measurements in 1986 (as built) and also in 1990 (4 years after construction). The small coefficient of variation of the measurements indicates somewhat the homogeneity of the porous pavement materials. Student's *t*-tests were performed to find the difference of the means of each measurement. At the 5 percent level of significance, significant differences were observed between all sets of data. From Table 5 it is apparent that, on average, there is a 2- to 3-pcf increase in the density of porous pavement asphalt concrete after 4 years in service. Although the average unit weight in 1990 was similar for both the outer wheelpath and between-wheelpath locations, the range in values suggests that some densification has occurred in the wheelpaths.

### Permeability

Permeability tests were conducted on the cores using a constant head water permeability test procedure. Table 6 summarizes the test results along with the core unit weights. The coefficient of permeability during the mix design formulation was 200 ft/day. The permeability and porosity of the in-service porous pavement have decreased, but the values are well above the 26 ft/day required by design. It is to be noted that the average coefficient of permeability in the wheelpaths is less than that of the between-wheel locations.

### Marshall Stability and Flow

Marshall stability and flow values were determined on four cores from an outer wheelpath (OWP) in 1986 immediately after construction. In 1990 five cores from an OWP and seven cores from between wheelpaths (BWP) were tested. Results

TABLE 5 Nuclear and Laboratory Densities of Porous Asphalt Concrete

| Statistic | DENSITY (pcf) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | AS BUILT (1986) | | | | IN-SERVICE (1990) | | | |
| | Nuclear | | Laboratory | | Nuclear | | Laboratory | |
| | OWP | BWP | OWP | BWP | OWP | BWP | OWP | BWP |
| Mean | 118.5 | 116.3 | 116.9 | 113.6 | 112.4 | 121.9 | 120.1 | 119.8 |
| St. Dev. | 0.8 | 1.1 | 0.35 | 0.35 | 2.0 | 1.7 | 2.0 | 1.9 |
| C.V. (%) | 0.68 | 0.95 | 0.30 | 0.30 | 1.63 | 1.40 | 1.67 | 1.6 |
| number | 4 | 4 | 4 | 4 | 18 | 18 | 13 | 15 |

Note: C.V.: Coefficient of Variation (%).

TABLE 6 Permeability Test Results

| | 1986 (After Construction) | | 1990 (In service) | |
|---|---|---|---|---|
| | Unit Weight (pcf) | K (ft/day) | Unit Weight (pcf) | K (ft/day) |
| OUTER WHEEL PATH: | | | | |
| Mean | 123 | 154 | 120.0 | 76.0 |
| Std. Dev. | -- | -- | 1.93 | 44.1 |
| C.V. (%) | -- | -- | 1.60 | 58.3 |
| Range | | | 117-125 | 27-166 |
| BETWEEN WHEEL PATH: | | | | |
| Mean | | | 119.7 | 80.5 |
| Std. Dev. | | | 1.82 | 61.2 |
| C.V. (%) | | | 1.50 | 76.1 |
| Range | | | 116-122.6 | 14-264 |

show that, on average, there is a fourfold increase in the Marshall stability value since construction. There has also been an increase in flow value, which may be attributed to the flushing done in summer 1989.

### Resilient Modulus

The resilient modulus test was performed on cores from 14 locations in the outer wheelpath and from 16 locations between the wheelpath. Each specimen was sliced into two samples approximately 4-in. in diameter and 2.5 in. high to represent the two different lifts used in placing porous asphalt concrete. The resilient modulus test was performed on these samples at 77°F according to ASTM D4123. Each specimen was tested in two positions (90 degrees apart) and at three different loading frequencies (1, 0.5, and 0.3 Hz) with a load duration of 0.1 sec in all cases. The applied load was between 85 and 130 lb with the majority around 100 lb. The horizontal deformation was measured and the modulus was computed by assuming a Poisson's ratio of 0.35. The instantaneous resilient modulus was the same as the total resilient modulus in about 95 percent of the cases. Therefore, only total resilient modulus was used in the analysis. Table 7 shows the summary of the test results. The resilient modulus of the porous asphalt concrete mix has increased significantly. The average resilient responses of the porous asphalt mix at the outer-wheelpath and between-wheelpath locations are the same, as indicated by the similar average resilient modulus values at these locations.

### Gradation of Asphalt Concrete

The aggregates extracted from the cores of asphalt concrete were tested for gradation to detect any change that might have occurred because of degradation of the open-graded aggregates after 4 years of service. Comparison of the results of the gradation analysis for the 1990 sampling of porous pavement, the as-built, and specified gradation for porous asphalt concrete showed no signs of degradation.

### Properties of Extracted Asphalt Cement

The extraction was performed in accordance with ASTM D2172. The extracted asphalt was then recovered by the Abson method (ASTM D1856). Penetration (ASTM D5) and absolute viscosity (ASTM D2171) of the extracted asphalt cement were determined. Comparison of the results with those parameters determined immediately after construction showed that there has been an increase in the viscosity of the asphalt cement.

### ATB

ATB samples were tested for extraction, moisture content of asphalt concrete, asphalt content, gradation, Abson recovery, penetration, and viscosity following ASTM D2172, D1461, D4125, C136, D1856, D5, and D2171, respectively. Usually the samples at consecutive station locations were combined to get enough recovered asphalt to perform the penetration and viscosity tests. Samples were not combined until the extraction, moisture content, and gradation had been measured. Results showed that the gradation of ATB aggregates was well within the as-built specified gradation.

### Open-Graded Subbase

The moisture content of the subbase samples was determined according to ASTM D2216. The samples were then tested for gradation as per ASTM C136. The gradation analysis results showed that there was no change in gradation of subbase materials 4 years after construction.

### Subgrade

Moisture content measurements were conducted on subgrade soils in accordance with ASTM D2216. The average moisture content at the outer-wheelpath locations and between-wheelpath locations was 4.7 and 1.5 percent, respectively. This was well below the optimum moisture content of 13 percent used during the design phase.

### Distribution of Moisture in Pavement

The moisture content of each layer of porous pavement was analyzed. No unusual moisture was detected in any layer.

**TABLE 7 Summary of Resilient Modulus Test Results: 77°F, 1 Hz**

| Statistic | RESILIENT MODULUS (ksi) | | | | |
|---|---|---|---|---|---|
| | 1986 Test | 1990 Test | | | |
| | OWP | OWP | | BWP | |
| | | Top | Bottom | Top | Bottom |
| Mean | 162 | 1004 | 888 | 962 | 986 |
| Std. Dev. | 38 | 271 | 287 | 232 | 359 |
| C.V.(%) | 237 | 27 | 32 | 24 | 36 |
| number | 6 | 14 | 14 | 16 | 16 |

## CONCLUSIONS

The porous pavement test section has performed satisfactorily for 5 years. Although a slight decrease in the infiltration rate has occurred, both the infiltration rate and the storage capacity are above the design values. The storage capacity of the pavement subbase and trench drain system has been underutilized. This might suggest an overly conservative design. If a design intensity storm occurs during the remaining service life, this should be verified. Visual observation during rain storms has shown that the surface of the porous pavement section does not include sheet flow, which provides a marked difference in stripe delineation and pavement glare during nighttime inclement weather driving as compared with conventional pavement. Mu-meter skid test results for the porous pavement section are comparable with those of conventional pavements (control). It was expected that the porous pavement would have higher skid resistance because of its improved drainage characteristics. This expected higher skid resistance may likely exist with heavier concentrations of water than that applied during mu-meter skid testing.

Several problems developed during the construction of the pavement structural section. Stability problems within the open-graded subbase and ATB layer resulted in serious rutting and a loss of section at the roadway median. A thin overlay was required during construction to restore the roadway surface to proper grade and riding qualities. Even with the additional overlay, the Mays roughness values are higher for the porous pavement than for the conventional pavement. No significant increase in roughness has occurred with time and use since the project was completed. Future design should consider using only stabilized materials with additional examination of the stability of the ATB layer.

Although a detailed analysis of the cost-effectiveness of the porous pavement section was not performed for this paper, the authors do not believe that this section provided significant, if any, cost savings over conventional design. True construction costs are seldom achievable with experimental sections.

Materials tests conducted on the pavement components indicate that the Marshall stability, resilient modulus, and asphalt cement viscosity of the open-graded asphalt concrete have increased with time. No cracking or significant surface deformation have occurred during the 5 years of service.

Annual FWD testing was conducted to establish the changes in layer properties. To date, little change has occurred in the layer moduli except for the open-graded subbase, whose modulus has decreased with time. This phenomenon is unexplained at present. Because these moduli were obtained through backcalculation analysis of FWD data, it is not certain whether the results are an artifact of the backcalculation procedure or true properties of the material.

No unusual moisture was detected in any layer of the pavement system. The subgrade moisture content has achieved equilibrium and less than optimum moisture content determined during the design process.

## RECOMMENDATIONS

It is recommended that the porous pavement be monitored for another 5 years to establish its performance characteristics. The performance monitoring scheme should include biennial Mays roughness, mu-meter, rut depth, and FWD deflection data collection. Visual distress surveys are recommended every other year. Laboratory tests of the cores and samples taken from this section should be conducted at the seventh and the tenth years. Correlation of the test results with observed performance is recommended.

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# Field Performance of Open-Graded Drainage Layers

J. J. Hajek, T. J. Kazmierowski, H. Sturm, R. J. Bathurst, and G. P. Raymond

The results of a field study carried out to investigate the performance of pavements incorporating open-graded drainage layers (OGDLs) are presented. OGDLs are used to ensure rapid and effective drainage of pavement structures and thus to improve pavement performance. Altogether, five paving projects built since 1975 and incorporating OGDLs are described and evaluated. The projects encompass flexible, composite, and rigid pavements, and include both asphalt-cement-treated and untreated OGDL materials. The evaluation was done in terms of (a) pavement performance evaluation, with emphasis, where possible, on comparing adjacent sections with and without OGDLs; (b) occurrence and severity of major pavement distresses investigated by coring and trenching; (c) assessment of in situ drainage done by observing movement of water within the pavement structure; and (d) laboratory analysis of excavated OGDL materials. The results of this study show that the existence of OGDLs alone does not guarantee a better pavement performance. This can be achieved only if the OGDL is a part of a properly designed internal drainage system.

Excess water trapped in the pavement structure can accelerate pavement damage and cause premature pavement failure. In order to remove the excess water, a number of highway agencies have built or are experimenting with free-draining layers or blankets using permeable granular materials (*1–3*). This permeable layer is placed between the pavement surfacing materials and the subgrade and is referred to herein as an open-graded drainage layer (OGDL).

Since 1975, the Ministry of Transportation of Ontario (MTO) has incorporated OGDLs into a number of different paving projects encompassing flexible, rigid, and composite pavements. Five of these projects were built before 1989; an additional three or four projects have been built since or are under construction. The objective of this paper is to review our experience with the five older projects and evaluate the influence of OGDLs on pavement performance. The purpose of this work is twofold:

1. To provide feedback for pavement design practices incorporating OGDLs (*4*). For example, to assess the need for OGDLs, their placement, and physical properties including gradation and protection of OGDLs from infiltration and clogging.
2. To determine if the function and performance of OGDLs can be predicted using the existing design procedures.

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## LEARNING FROM THE PAST

The most recent OGDL project evaluated herein was built in 1987 and was probably designed 1 or 2 years before. Since then, our design practices and understanding of what constitutes an appropriate pavement subdrainage design have changed considerably. For example, in the past, the OGDLs were usually not connected directly to longitudinal subdrains, whereas current designs ensure a direct connection between the two elements. The question can be posed: Why study the performance of pavement subdrainage designs that are old and are unlikely to be built again?

The reason for studying the performance of old designs is that we can always learn by evaluating and verifying our theoretical understanding of material behavior and pavement performance using field observations. Although the materials of drainage layers and their arrangement can always change, the basic design principles are universally applicable and change only if they do not adequately explain what happens in the field. In other words, we are studying the field performance to verify our design principles.

## GENERAL CONSIDERATIONS IN SUBDRAINAGE DESIGN

Pavement systems, particularly those containing granular base and subbase materials, should not contain free, gravitational water (water that moves because of gravitational force). If free water is present, the effects of dynamic traffic loading can lead to pumping (movement of free water and suspended aggregate particles underneath two adjacent pavement slabs) and to premature loss of riding quality. There is also a concern with the loss of stability due to high pore pressures in saturated dense-graded aggregate layers.

The major design considerations include the permeability and thickness of the OGDL, the collection system, and the protection system. The recommended permeability for OGDLs carrying water across a pavement roadbed on a 2 or 3 percent slope is typically above 1.7 mm/sec (500 ft/day) (*5*). Ridgeway (*6*) implicitly recommends 3.5 mm/sec (1000 ft/day). At any rate, the typical gradation curves for OGDL materials used on past Ministry projects indicate that our materials meet and significantly exceed the recommended permeability criteria (*7*).

## FIELD EVALUATION OF OGDLs

In this section we describe pavement structure and subdrainage design of the five projects incorporating OGDLs and evaluate their performance. The evaluation is done in terms of

1. 1990 pavement performance evaluation, with emphasis, where possible, on comparing the performance of the adjacent pavement sections with and without OGDLs;
2. Occurrence and severity of predominant pavement distresses investigated by taking 150-mm-diameter cores and by trenching;
3. Assessment of in situ drainage by observing movement of water within the pavement structure; and
4. Laboratory analysis of OGDL materials obtained by coring, excavating, or both. The analysis included tests for asphalt cement (AC) content (ASTM D 2172), grain size distribution (ASTM C 127-84), bulk density (ASTM 1188-83), and other routine tests. The results are summarized elsewhere (*8*). Only selected results are given in this paper.

The main characteristics of the five projects incorporating OGDLs are summarized in Table 1.

### Highway 56

The site encompasses four sections: three test sections with OGDLs and a control section without an OGDL (Table 2). It may be noted that all four sections have the same design thickness of asphalt concrete (130 mm) and the same total thickness of granular materials (535 mm, considering OGDL materials as granular materials). However, on the basis of field investigation by coring and trenching, it appears that the actual asphalt concrete thickness is somewhat higher on sections with untreated OGDLs. This may be attributed to the unevenness or distortion of the untreated OGDLs before paving.

The untreated OGDLs were placed full width across the roadbed with daylighting at the ditch slopes below the shoulder rounding. The AC-treated OGDL was placed below the pavement only and the untreated OGDL was placed on the remaining portion of the roadbed below the shoulders and again daylighted. Where possible, OGDLs were placed on a 3 percent crossfall (*9*).

#### *Pavement Performance Evaluation*

After 15 years of service, all four sections exhibit quite similar performance, both in terms of riding comfort rating (RCR = 6.0) and the occurrence and severity of pavement surface distresses. The main distress (on all sections) is frequent severe, full-width transverse cracking. The transverse cracks are often cupped (the depression is about 12 mm on 300 mm straightedge); some are starting to become multiple (transverse cracks) and some to alligator. Sealant bond failure is quite frequent.

#### *Distress Investigation by Coring and Cutting*

The investigation by coring (taking 150-mm wet cut cores) and cutting (cutting 600- × 250-mm slabs from the pavement surface) focused on transverse cracks, which are the main distress and the major reason for upcoming pavement rehabilitation.

The OGDL material obtained at locations far from cracks appeared to be undamaged and dry. The AC-treated OGDL material (Section 3) was usually recovered in one piece and was bonded to the overlying asphalt concrete. Its AC content was close to the 3 percent value specified for this section (see Table 1). A quite different situation was observed near transverse cracks where the OGDL material was wet and contaminated with fines from the base material (Granular A).

A composite representative condition of transverse cracks based on coring and cutting is shown in Figure 1. It appears that the existence of OGDLs had no significant influence on the frequency of transverse cracks or on the condition of transverse cracks in terms of crack opening, asphalt concrete stripping at the bottom of the asphalt concrete layer, or cupping. There was noticeable stripping of asphalt cement from the aggregate particles of the treated OGDL material. A low

**TABLE 1 MTO Projects Incorporating Open-Graded Drainage Layers**

| Location | Date of Constr. | Contract No. | 89 Traffic | | No. of Traffic Lanes | Pavement Structure | Length of ODGL Sect. (m) | Asphalt Content of ODGL (%) | Thickness of OGDL mm |
|---|---|---|---|---|---|---|---|---|---|
| | | | Volume (AADT) | Truck* % | | | | | |
| Hwy. 56, south of Elfrida | 1975 | 75-02 | 8400 | | 2<br>2<br>2 | Flexible | 122<br>122<br>122 | 0<br>3<br>0 | 75<br>75<br>150 |
| Hwy. 3N, west of Leamington | 1982 | 82-01 | 8500 | 9.5 | 2 | Rigid | 5200 | 2 | 100 |
| E.C. Row near Dominion Blvd. | 1987 | 87-37 | 25000 | 12 | 4 | Composite | 500 | 2 | 100 |
| E.C. Row, near Huron Church Rd. | 1987 | 87-23 | 11000 | 13 | 4 | Composite | 300 | 2 | 100 |
| Hwy. 16N, south of Ottawa | 1984 | 84-22 | 11000 | 10 | 2 | Flexible | 1000 | 2 | 100 |

* *Estimated*

**TABLE 2 Arrangement of Test Sections on Highway 56**

| Test Section | Thickness of pavement layers, mm | | | |
|---|---|---|---|---|
| | Asphalt Concrete | | OGDL (Observed)[1] | Granular A (Design) |
| | Design | Observed[1] | | |
| 1 Control | 130 | 140 | 0 | 535 |
| 2 75 mm untreated OGDL | 130 | 175 | 75 | 460 |
| 3 75 mm A.C. treated OGDL | 130 | 140 | 75 | 460 |
| 4 150 mm untreated OGDL | 130 | 175 | 150 | 385 |

*Notes: [1]Established by coring or cutting (trenching)*
*The total thickness of granular materials, considering OGDL to be a granular material, was 535 mm on all sections.*

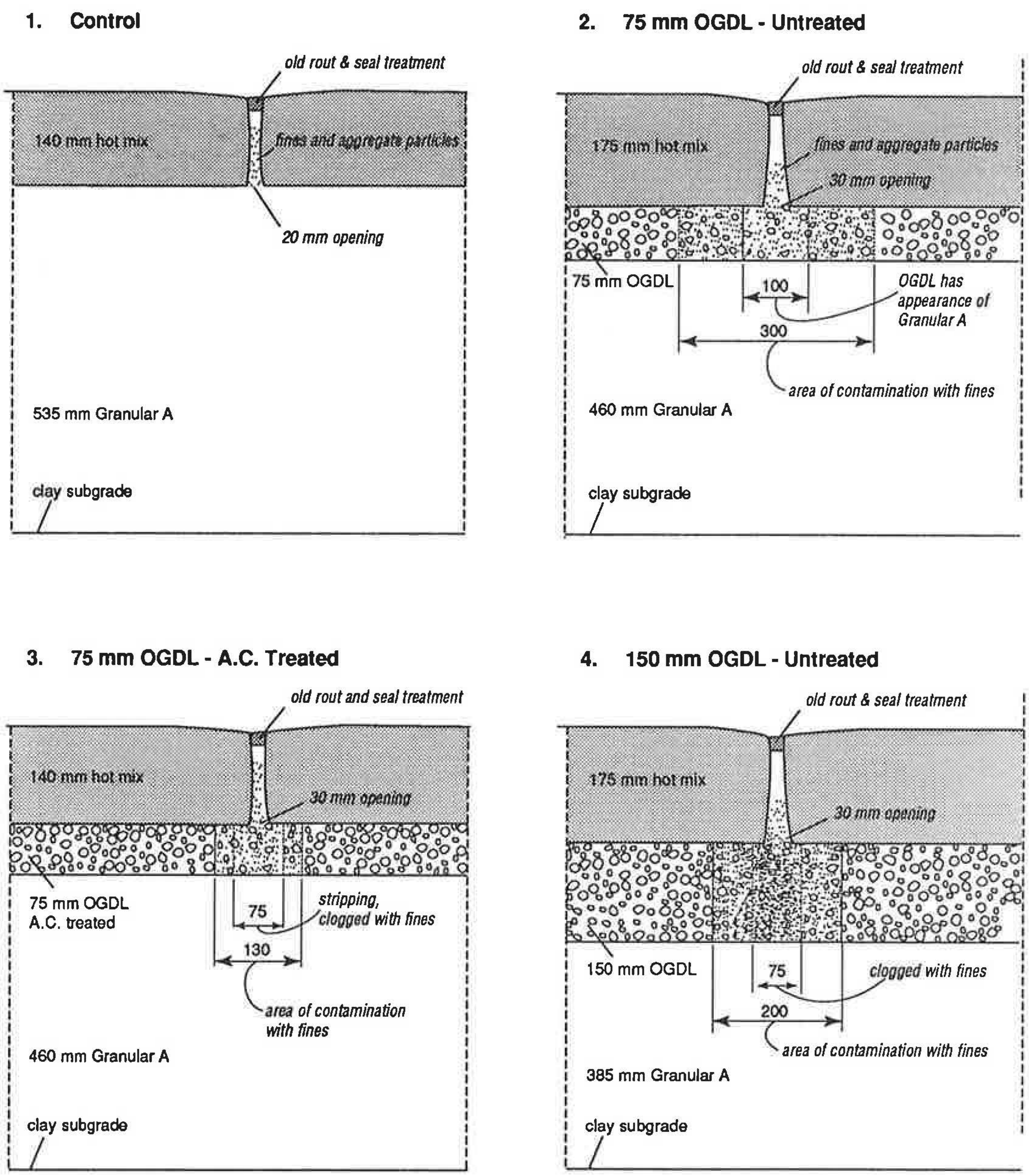



**FIGURE 1 Typical condition of full transverse cracks, Highway 56.**

value of 1.3 percent AC was obtained from a sample taken below a fully developed transverse crack. Otherwise the type and the area of contamination with fines and the amount of fines in the OGDL materials were quite similar on all three OGDL sections.

### *Investigation of In Situ Drainage*

**Daylighting of OGDL** The topsoil on the ditch slope was removed at several locations to examine the condition of the OGDL material. The topsoil cover was about 300 mm thick. The OGDL was distinguishable from the surrounding layers of Granular A material even though it was contaminated with topsoil and Granular A material. Underneath the shoulder, the OGDL material was less contaminated and appeared to be free draining.

**Crossfall Drainage** Water was poured into a 600- × 250-mm cut in the asphalt concrete, and the OGDL material and its propagation towards the ditch were observed using two additional openings between the cut and the ditch daylighting. As indicated by the results given in Figure 2, the crossfall drainage appeared to be sufficient at the location investigated, with an average horizontal permeability of 10 mm/sec (2800 ft/day). This exceeds the recommended minimum by a factor of 3.

**Permeability** Water was poured into the core holes and cuts, and the drop in the water level was observed. The areas of contamination shown in Figure 1 drained very slowly. Usually, there was no observable water drop in 10 min. Noncontaminated areas drained so fast that no water head could be maintained using a 25-L container.

## Highway 3N

The pavement section with an OGDL on Highway 3N is part of a large 1982 experiment investigating the design, construction, and performance of plain-jointed portland cement concrete (PCC) pavements. The experiment encompasses four different pavement designs, including one design with an OGDL (Table 1). The design and performance of all four pavement sections have been described in detail elsewhere (*10*) and will be only briefly reviewed here.

The pavement structure of the four sections, built on a silty-clay subgrade, is summarized below and the arrangement of the OGDL section is shown in Figure 3.

- Section 1: 305-mm plain jointed PCC slab.
- Section 2: 203-mm plain jointed PCC slab, 100-mm open-graded drainage layer.
- Section 3: 178-mm plain jointed PCC slab, 127-mm lean concrete base.
- Section 4: 203-mm plain jointed PCC slab, 127-mm lean concrete base.

The longitudinal drainpipes (installed on all four sections) were plowed in and backfilled with the excavated shoulder material. The highway grade in the vicinity of the OGDL section is mainly at grade.

### *Pavement Performance Evaluation*

A detailed pavement performance evaluation of all four pavement test sections was conducted after 6 years (*10*). Briefly, all four PCC sections have roughly similar pavement roughness. However, the OGDL section and the full-depth PCC section have considerably less cracking distress, approximately five to eight cracks per kilometer, than the remaining two sections with lean concrete bases, which have approximately 60 to 70 cracks per kilometer. Much of the cracking in the lean concrete base sections is related to reflective cracking (from the lean concrete base). It is interesting to note that

1. The OGDL section is marginally less expensive than the full-depth section, but provides comparable performance, and
2. The structural strength of the OGDL section is lowest of the four.

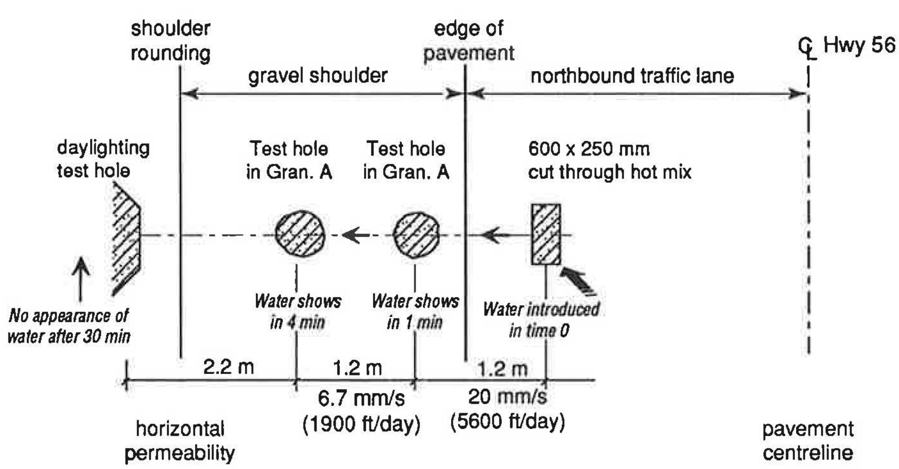
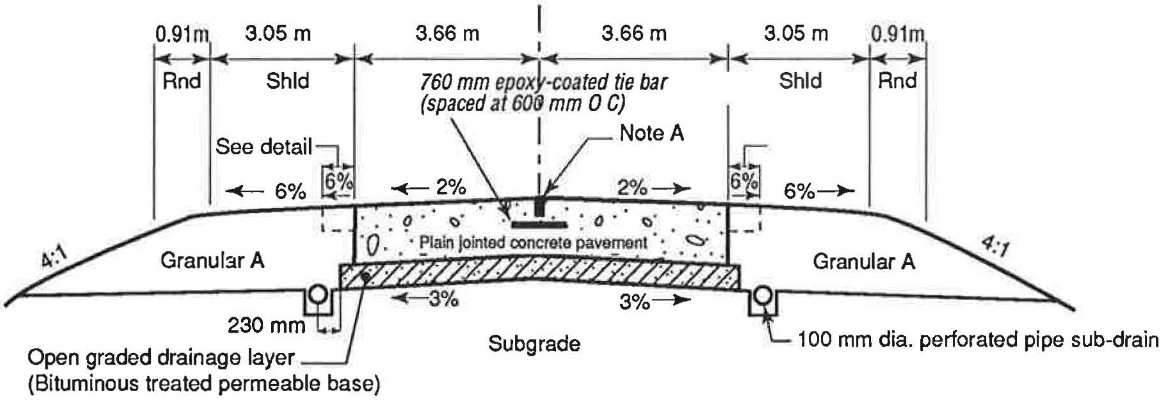
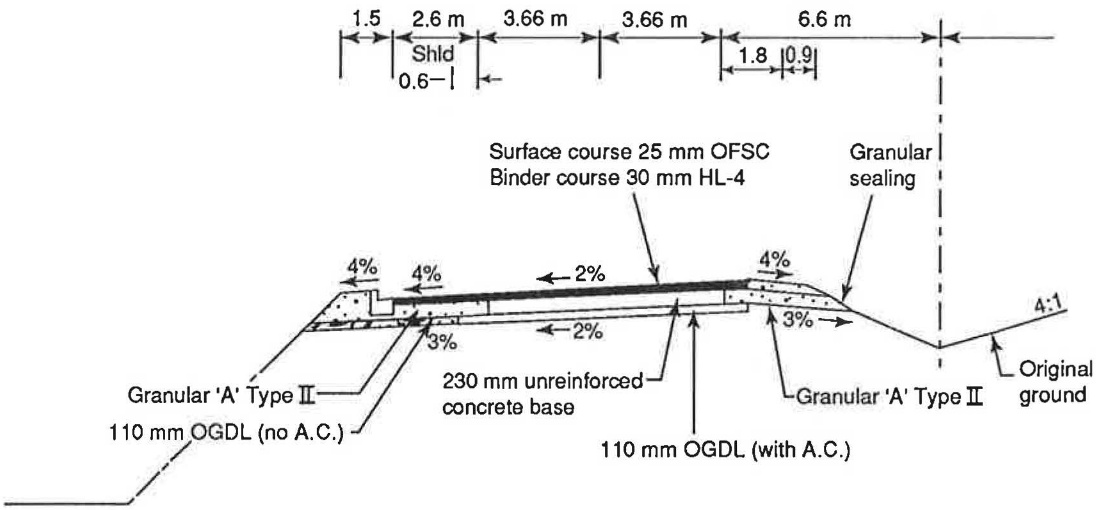


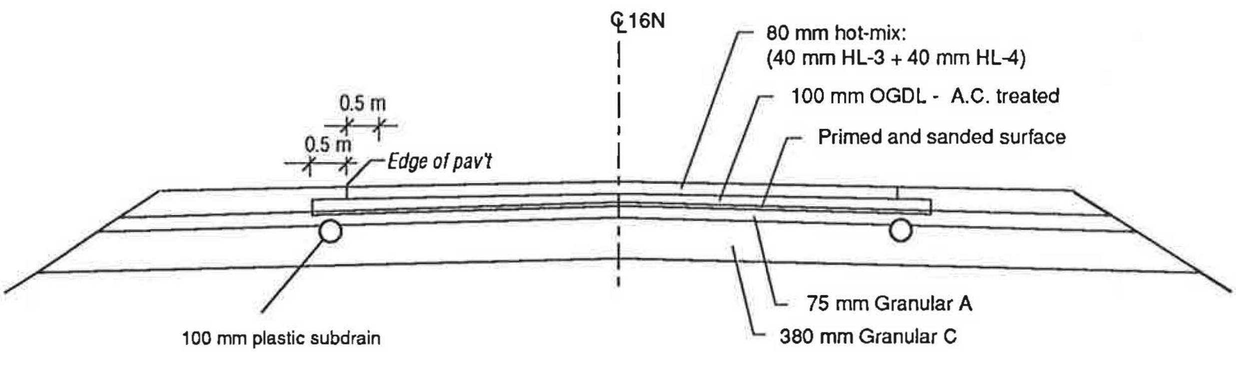

**FIGURE 2 Investigation of crossfall drainage, Highway 56, Section 3.**




A/ OGDL section on Hwy 3N




B/ OGDL section on E.C. Row Expressway




C/ OGDL section on Hwy 16N

**FIGURE 3 Typical OGDL sections.**

The major distresses on the OGDL section are faulting of transverse joints and corner cracking. The transverse joints are stepped, on average, by about 3 mm, which is common to all sections. The maximum stepping is about 8 to 12 mm.

*Distress Investigation by Coring*

The coring was done in June 1989, following a night of heavy rainfall. Before the coring started, pumping action of traffic at transverse joints was clearly observed: small droplets of water were ejected from the joints by passing trucks and squirted upwards, making the pavement surface close to the joints wet and stained with clay and silt particles.

The cores taken at mid-slab (far from the joints) revealed a well-preserved OGDL material. About 80 percent of the material was recovered from the 175-mm bit cylinder in one piece and was bonded to the PCC. It was observed that the OGDL material in the core holes was flooded with water and that soon after the core was removed, the water level rose to within 50 mm of the pavement surface. The bottom 20-mm layer of the OGDL material was contaminated with subgrade (silty clay) and was loose showing signs of stripping. The AC content of the bottom portion of the core was 2.0 percent compared with 2.7 percent obtained for the top portion of the core.

The cores taken at faulted transverse joints near the center of the lane and the outside wheelpath revealed loose, partly stripped OGDL aggregate (AC content ranging from 1.1 to 1.9 percent) heavily contaminated with subgrade material. The OGDL material in all core holes was flooded. There was no loss of material at the bottom on the PCC slabs. Figures 4 and 5 contrast the condition of the OGDL material obtained at a mid-slab location with that close to a faulted transverse joint.


**FIGURE 4 150-mm core obtained from cracks or joints, Highway 3N. OGDL material basically in one place and partially bonded to surface layer.**

**FIGURE 5 150-mm core obtained at transverse joint, Highway 3N. OGDL material loose mixture of stripped OGDL aggregate particles and subgrade soil.**

*Investigation of In Situ Drainage*

Approximately 12 hr after a heavy rainfall, the OGDL was saturated with water at the locations investigated. There was no noticeable drop of the water level during an hour. It appears that the water ponding in the OGDL is caused by relatively impervious granular materials separating the OGDL from the lateral drains and ditch daylighting [Figure 3(*a*)]. Another factor contributing to ponding may be a low hydraulic gradient of the lateral subdrains, which are covered with a geotextile material protecting the perforated plastic subdrain from contamination.

Outlets of the lateral subdrains located on the ditch slopes were in good condition, relatively clean and free of vegetation. Most of them were discharging water, but not in the amounts expected. The rate of water discharge from the outlets seemed to be the same on all four sections. This highlights the need for a drainage collection system in direct contact with the OGDL.

**E.C. Row Expressway in Windsor**

The original composite pavement design, shown in Figure 3(*b*), was constructed at both locations on E.C. Row Expressway. The OGDL material was placed on the full width of the roadbed with daylighting at ditch slopes. The AC-treated OGDL material was placed underneath the pavement and extended 600 mm beyond the edge of pavement; the untreated material was placed underneath the remaining width of the shoulder.

*Pavement Performance Evaluation*

The pavement performance of the two OGDL sections is similar. The riding comfort rating is 8.5 (out of 10) and the predominant distress is full-width slight transverse cracks reflected above contraction joints that were precut in the plain PCC base. The transverse cracks have reflected above nearly all contraction joints, resulting in 3- to 6-m spacing. Some of the cracks are starting to spall, particularly on driving (truck) lanes. No crack stepping was observed; however, some rocking of the slabs was noticeable between two adjacent slabs separated by a spalled transverse crack when loaded trucks were passing.

### *Distress Investigation by Coring*

The majority of cores were taken in the vicinity of transverse cracks. One core was taken near the middle of the (paved) right shoulder. The results of coring are shown in Figure 6. In many respects the results are similar to those observed on Highway 3N. The OGDL material at locations far from transverse cracks was in a very good condition with little or no stripping and some contamination by the subgrade material at the bottom. The level of contamination and stripping increased with closeness to transverse cracks. There was no loss of PCC material at the bottom of the PCC base near the contraction joints.

### *Investigation of In Situ Drainage*

**Daylighting** The removal of granular material and topsoil on the ditch slope (i.e., digging of daylighting test holes) revealed a well-preserved layer of the OGDL material. At the daylighting test holes close to transverse cracks, the OGDL material was quite wet and the underlying subgrade material was saturated. It appears that water is entering the pavement structure through cracks and drains through the OGDL.

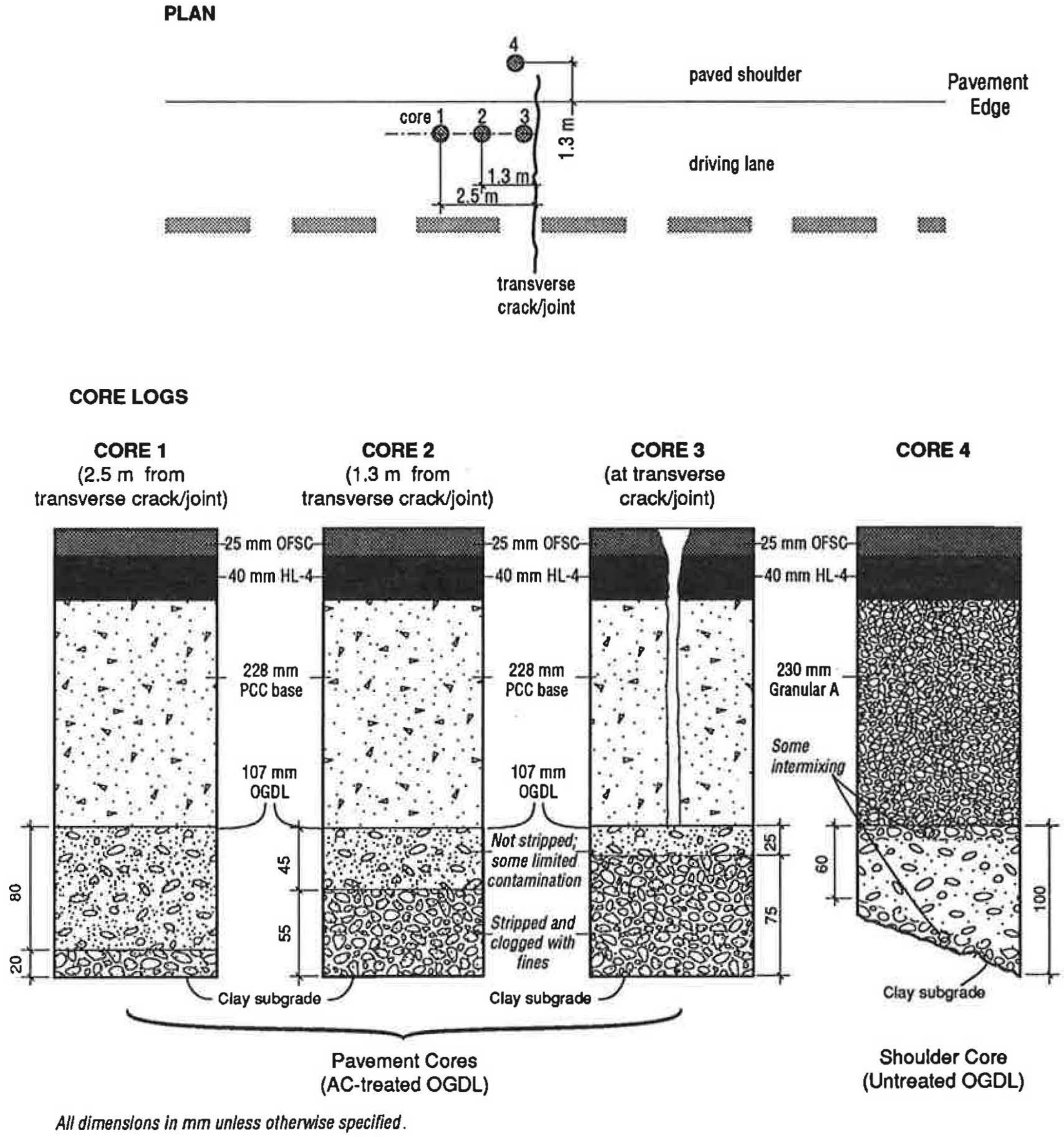



**FIGURE 6 Condition of OGDL Material, E.C. Row Expressway.**

**Crossfall Drainage** Water was poured into a 150-mm core hole drilled in the middle of the paved shoulder (Core 4, Figure 6). The rate of water dissipation was quite high (about 0.2 mm/sec), but because of the overall wet subgrade condition and the 3 percent longitudinal pavement grade, it was not possible to observe additional water seepage at the daylighted test hole or holes. The log of Core 4 in Figure 6 also shows uneven thickness of the untreated OGDL beneath the shoulder and some minor contamination.

**Permeability** The in situ permeability was tested by pouring water into core holes and measuring the rate of water dissipation. The results suggest that only the top portion of the OGDL was free draining in the vicinity of transverse cracks (7). Because of heavy contamination with subgrade fines, the bottom portion of the OGDL was quite impervious.

The two OGDL sections will benefit from an early routing and sealing of transverse cracks, which should prevent free water from entering the pavement structure.

### Highway 16N

Three different sections—an OGDL section, a conventional section, and modified granular section—were built in 1984. The structure of the OGDL section is shown in Figure 3(*c*). The conventional section consists of 130 mm of asphalt concrete, 150 mm of gravel base, and about 425 mm of sand subbase. The modified granular section consists of 120 mm of asphalt concrete, 75 mm of gravel base, and about 300 mm of sandy subbase. All unbound granular materials in this section were modified by removing most of the fine aggregate particles. Only the OGDL section has lateral subdrains as shown in Figure 3(*c*).

#### *Pavement Performance Evaluation*

The roughness of all three sections is quite similar, as measured by the Portable Universal Roughness Device, whereas the pattern of cracking distresses is somewhat different. In addition to slight longitudinal cracks (midlane, center line, and wheel track cracks), which are found on all three sections, the conventional section has a noticeably higher density of these cracks and also has a few slight half and full transverse cracks. It was also reported that pavement roughness on the OGDL and modified granular design sections, as measured by in-vehicle ride, does not increase during winter, although it is noticeably higher on the conventional design section.

It is still too early to estimate the difference in the lifespan of the three designs. Nevertheless, in addition to its better performance, the OGDL section has also lower granular base equivalency (GBE) thickness (588 mm) than the conventional section (660 to 685 mm). The calculation of the GBE thickness, which relates all materials to an equivalent thickness of crushed gravel (Granular A), assumed that the GBE factor of the OGDL material is equal to 1. The OGDL section was also reported to be somewhat less expensive to construct than the other two sections.

#### *Investigation of Distresses by Coring*

The coring revealed undamaged OGDL material even if the cores were taken at (longitudinal) cracks. The OGDL material was usually well bonded to the bottom of the AC layer. The primed and sanded surface of the Granular A material [Figure 3(*c*)] appeared to be undamaged and in very good condition.

The thickness of the OGDL ranged from 70 to 110 mm and its AC content varied from 1.3 to 2.0 percent. Below the shoulder, where the OGDL material was more exposed to moisture, the AC content ranged from 0.7 to 1.1 percent. On the basis of tests conducted on five cores, the average bulk density was 1852 kg/m$^3$, with a range of 1791 to 1894 kg/m$^3$. There was no obvious reason for the spread in the bulk density values that could be related to sample location.

#### *Investigation of In Situ Drainage*

**Crossfall Drainage** Water was poured into core holes in the pavement and its progression toward the ditch was observed by using test pits dug in the granular shoulder. The sketch of the experiment given in Figure 7 shows that the crossfall drainage was impeded. A direct connection between the OGDL and the collection system is not provided. The OGDL material is abutted by Granular A material, and the longitudinal subdrains are separated from the OGDL by the primed and sanded surface of the Granular A layer. According to Figure 7, the water that traveled 2 m through the Granular A material on a 3 percent crossfall was detected in 20 min; the water that traveled the same distance through the OGDL on a smaller slope was detected in 1 min.

**Permeability** No significant water head could be established when water from 25-L containers was poured into 150-mm core holes. It appears that water dissipated very quickly over the primed and sanded Granular A layer.

**Subgrade Moisture Content** The effect of the OGDL on reducing the moisture at the pavement-subgrade interface was evaluated by time domain reflectometry based moisture gauges

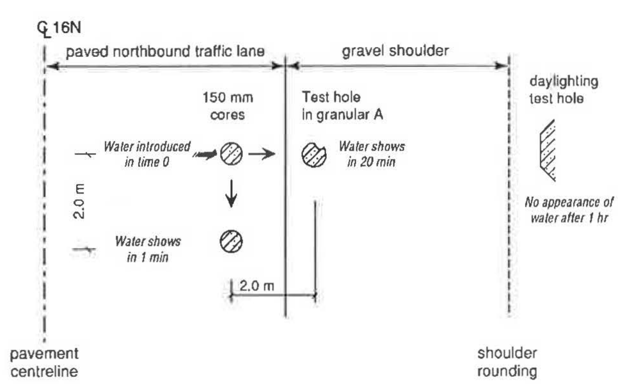



FIGURE 7 Investigation of crossfall drainage, Highway 16N.

(*11*) installed during construction on all three sections. The gauges were installed on the top of the subgrade (*a*) below the edge of pavement and (*b*) below the center line of the northbound lane. The results summarized by Hajek et al. (*7*) indicate that the presence of the OGDL may have somewhat reduced subgrade moisture content.

## APPLICATION OF DESIGN PRINCIPLES

### Intrusion of Fines

One of the major problems limiting the performance of OGDLs, identified by the field investigation on Highways 56 and 3N and the E.C. Row Expressway, was clogging of the OGDL material with fines. Movements of the fine soil particles can be caused by the movement of water from a fine-grained soil (such as subgrade clay) to the OGDL, by the pumping action of repetitive axle loading, and by penetration of the coarse OGDL material into a fine-grained material during the period of spring thaw. Generally recommended criteria for the design of protective granular filters (*5*) are used in this section for after-construction prediction of the clogging phenomenon.

#### *Fines from Subgrade*

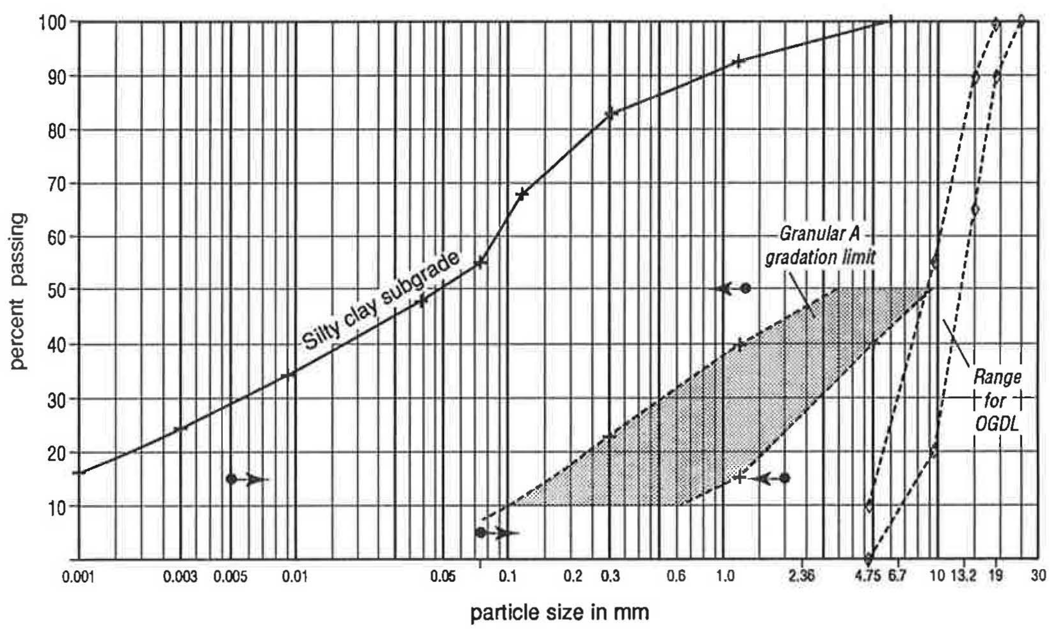
An arrangement similar to that existing on Highway 3N and on the E.C. Row Expressway is considered, in which the OGDL is placed directly on a clay subgrade. Representative gradation curves for the OGDL material and a silty-clay subgrade are given in Figure 8. Also shown as arrows in Figure 8 are gradation criteria for a granular filter based on the report by Moulton (*5*). The gradation range of the OGDL material is completely outside the filter criteria, and it is obvious that a filter layer should be provided to protect the OGDL from clogging. Consequently, in the absence of a filter layer, the observed intrusion of fines in the field is to be expected.

Figure 8 also gives a gradation range for Granular A material, which satisfies the filter requirements for the protection of the OGDL from intrusion of fines from the subgrade and can be thus considered a filter. It now remains to be checked if the OGDL material needs protection from intrusion of fines from the Granular A material. This is done separately in the following section.

#### *Fines from Granular A Material*

An arrangement similar to that existing on Highway 56 is considered, in which the OGDL is placed atop Granular A material. The generally recommended filter criteria of Moulton (*5*) were satisfied and no filter was required. However, field observations suggest otherwise: the OGDL was clogged near the transverse cracks where pumping action of traffic took place. This suggests that caution must be exercised when pumping is a possibility.

In addition to granular filters, other measures to reduce the occurrence of clogging include curtailing the amount of fines in the granular base material, priming and sanding the top of the granular base, and the using geotextile filters.




*Note: Filter gradation should be within the limits indicated by arrows*

**FIGURE 8 Filter requirements to protect OGDL from intrusion of fines from subgrade.**

### Collection System

Another major problem limiting the performance of OGDLs was an inadequate collection system for rapid removal of water carried by the OGDL toward the pavement edge.

In the case of Highway 56 and the E.C. Row Expressway, there is no collection system per se: the removal of water depends on daylighting below shoulder rounding. The lateral drainage through daylighting may be restricted by

1. Contamination from overlying granular shoulder materials and topsoil, and blocking by the layer of topsoil;
2. Uneven or inadequate thickness of the OGDL, or both; an example of uneven thickness is shown in Figure 6 (for Core 4); and
3. Inadequate crossfall grade of the OGDL.

It may be noted that some authorities (*6*), as well as the MTO current guidelines, do not recommended the use of daylighting for draining of the OGDLs.

In the case of Highways 3N and 16N, drainage collection systems are in place, but the OGDLs are not directly connected to longitudinal subdrains. The high permeability of the OGDL material is not utilized if free water must overcome a relatively impervious barrier, for example, that formed by Granular A material, before it can reach lateral subdrains.

## CONCLUSIONS AND RECOMMENDATIONS

1. The existence of OGDLs as such does not automatically guarantee improved pavement performance. To achieve improved performance, a comprehensive internal drainage design harmonizing the use of OGDLs (in terms of permeability, thickness and crossfall gradient), a collection system (dimensions and placement of subdrains and outlets), and a protection system (type and placement of filters) is needed.
2. High-permeability OGDLs are susceptible to clogging by fines when they are subjected to pumping at cracks and working joints by repeated traffic loads, particularly if the collection system or the filter system, or both, is inadequate and free water is allowed to linger in the OGDL material. Generally accepted filter criteria may not be sufficient to protect high-permeability OGDL materials from clogging under these circumstances.
3. The clogging and contamination by fines can reduce the permeability of the OGDL material by several orders of magnitude.
4. Stripping of asphalt cement from the bituminous-treated OGDL material does not appear to affect performance of the layer. One hundred percent crushed aggregate is specified to ensure long-term stability of the layer, and the bituminous stabilization is used only to simplify construction.
5. Monitoring of the performance of pavement sections incorporating OGDL materials should continue along the lines outlined in this paper.

## ACKNOWLEDGMENTS

The authors wish to acknowledge the effort of all those who designed and constructed the pavement test sections investigated in this study. Their efforts and foresight made this paper possible. Valuable comments and information were received from Sam Cheng. Field work was greatly assisted by M. Stott and A. Weremi.

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# First Test on the CEDEX Test Track

RECAREDO ROMERO, AURELIO RUIZ, AND JAVIER PEREZ

The test track built by the CEDEX Road Research Center at El Goloso on the outskirts of Madrid is a step forward in the field of lineal tracks because it extends the length of the test area and also increases the load application speed. It was built in 1987 and came into operation in January 1988. A description of the installation and of the first test carried out are given, including the definition of parameters and criteria used and an initial analysis of the results obtained (surface evenness, surface cracking, and deflections). Finally, the general conclusions are given and the consequences of the test for the Spanish Design Catalogue are discussed.

The objective pursued with full-scale test tracks is to provide a tool that enables the passage of vehicles over a pavement to be simulated in an accelerated and controlled way. The lineal test tracks built to date simulate back-and-forth movement. In spite of the advantage of simulating linear movement, they have the drawback of short length (up to 12 m) and low speed (up to 30 km/hr) because the need for the corresponding acceleration and braking areas limits both parameters to a large extent.

The test track built by the Centro de Estudios y Experimentacion de Obras Publicas (CEDEX) at El Goloso on the outskirts of Madrid is a step forward in the field of lineal tracks. The length of the tests area and also the load application speed exceed those existing up to now. The track was built in 1987 and came into operation in January 1988; it is the largest test track with simulated traffic in the world today (Figure 1).

The first test demonstrated the way the installation functioned. Some improvements or modifications of its different elements were developed along with the data collection and analysis systems, so it has taken 3 years to apply 1 million loads. At the end of this period the installation was in perfect condition for constant use and two new vehicles are being built to optimize its performance.

An analysis of the results obtained in the first test is presented. This analysis does not include the results of the instruments placed on the pavement.

## DESCRIPTION OF INSTALLATION

### Infrastructure

The CEDEX test track consists of two straight stretches joined by two curves. The straight sections and the curves are all approximately 75 m long, giving a total length of track of 304 m in the average transverse path of the load simulator vehicle (Figure 1).

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The curved sections are not used for pavement tests. Their main application is for the testing of surface materials: paints, wearing courses, surface dressings, and so forth. The curves have therefore been given a pavement section that has a reinforced-concrete base course and a lean concrete subbase designed to withstand an indefinite number of tests without deteriorating. In this way it will be necessary to replace only the wearing course in successive tests.

Without the transition zones between the straight and curved sections, there is 67 m left for pavement structure tests in each straight stretch. Because 20 m is considered the minimum length for a test to have any significance, the track's testing possibilities include six sections, each approximately 20 m long (Figure 1).

Although the pavement on the curves rests on the ground, the test sections on the straight stretches are constructed inside U-shaped watertight boxes made of reinforced concrete. The purpose of this system is to isolate the behavior of the pavement from that of the surrounding ground, and it allows the subgrade to be flooded for testing under different groundwater conditions.

The boxes are 2.60 m deep, so that embankments can be built at least 1.25 m high. The 8-m width allows the use of conventional machinery and the usual road-building procedures.

On the inside perimeter of the oval band, a reinforced-concrete rail beam has been installed to serve as a guide for the traffic simulator vehicle and to permit total control of the load path. This rail beam is anchored in the straight stretches

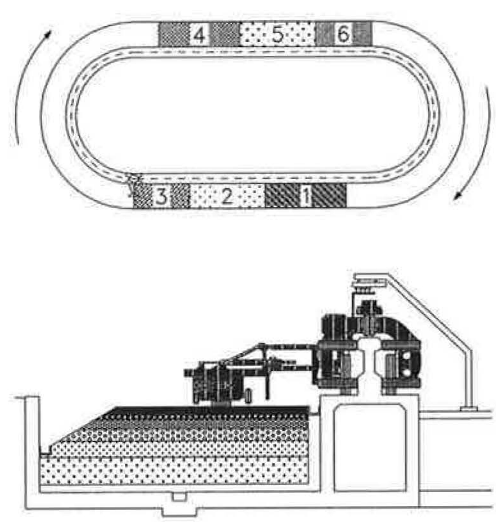


**FIGURE 1 Plan and cross section of test track.**

to underground, accessible galleries that house the connections of the permanent measuring equipment for each type of test (Figure 1).

Finally, a device has been installed that enables the test stretches to be covered, if desired; water sprinklers to simulate rain or any other mechanism to control the climatic conditions can be installed under the cover (Figure 2).

### Traffic Simulator Vehicle

The traffic simulator has two distinct parts: the guiding cart and the load cart (Figure 3). The latter applies the gravity stress. Its total weight (including the ballast) is 6.5 tonnes, the equivalent of 13 tonnes/axle, which is the maximum permitted for single axles in Spain. The load is applied by a pair of twin wheels with conventional tires and an inflated pressure of 8.5 kg/cm$^2$. This cart is self-propelling and therefore generates the movement of the whole unit. The motor is electric and runs on power from the overhead rail above the rail beam.

The vehicle has tires and suspension similar to those used by heavy road vehicles. It is possible to apply single or twin wheels. The design speed during the tests is 30 to 45 km/hr and the maximum speed is 50 km/hr.

The load cart can travel crosswise because of the action of a pneumatic jack incorporated in the vehicle. The maximum shift is $\pm 400$ mm, which produces a tread strip 1.3 m wide. The crosswise change in position occurs automatically by means of a centralized system that programs the control of the vehicle. The distribution of the passes follows a standard curve responding to actual distributions measured on roads.

The guiding cart runs along the concrete rail beam and is coupled to the load cart. Both are joined by articulations, allowing the load vehicle to move in a normal plane in the direction of movement. The articulation consists of two bogies mounted on the ends of a metal beam with two turning crown gears, which enable it to adapt perfectly to changes in path in the curved sections.

There are two cupboards on the guiding cart beam, which contain the alarms and control mechanisms; the compressed air generating equipment, which keeps the load wheel pressure constant and serves the pneumatic suspension and braking when the driving axle needs them; and the power switch trolleys fitted with oscillation-absorbing devices.

The control center, in a small building inside the loops of the track, automatically controls the movement of the vehicle (speed, transverse path change, etc.) and the orders communicated by radio. In addition, countless alarm sensors have been fitted into the vehicle itself to detect any anomaly that may occur during the test. If any problem occurs, the vehicle stops and the time and cause of the breakdown are stored in the control computer. This enables automatic unmanned functioning to take place with no risk to the installation and the track to be used continuously.

**FIGURE 2 General view of test track.**

**FIGURE 3 Vehicle traffic simulator.**

## FIRST TEST

### Pavement Section

The sections tested cover two different treatments for the same traffic situation (50 to 200 heavy vehicles per day) and subgrade [California bearings ratio (CBR) between 10 and 20] in accordance with Spanish Design Standards 6.1. and 6.2-IC. One treatment uses a large thickness of granular material (25 cm of crushed rock plus 25 cm of gravel) under an average 15-cm layer of asphalt mix. The other uses a much thinner layer of granular material (25 cm of crushed rock) and an increased asphalt layer (18 cm). In the sections located in the remaining four substretches, the thickness and type of lower layers remained the same, with the only change being in the thickness of the asphalt layer. The sections used in the first track test are shown in Figure 4.

The objectives of the test were to reach conclusions on

1. Which of the two standard sections of the Design Catalogue provided a better service life, and
2. The relative decrease in service life of these sections for each centimeter less of asphalt layer.

### Materials

At the bottom of the embankments and in the side walls of each box, a drainage material was installed, separated from the ground or pavement by a geotextile. The scale of thicknesses was introduced into the ground at the top level of the subgrade. The pavement was laid on a subgrade with a CBR

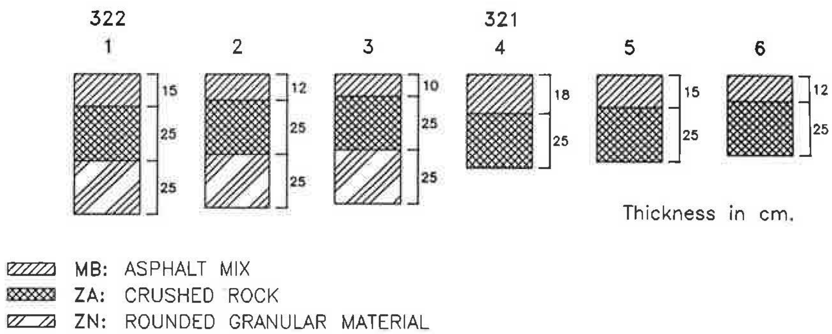


**FIGURE 4 Sections tested.**

of 10. Table 1 gives the characteristics of the gravel and crushed limestone rock used.

The asphalt mixes used were of two types following customary practice. The 5-cm-thick wearing course contained coarse siliceous aggregate with a maximum grade of 12 mm. The intermediate layer contained coarse limestone aggregate with a 20-mm maximum particle size. The thickness of this layer varied in the different stretches between 5 and 13 cm and was laid in two passes in the thickest test sections.

The characteristics of the asphalt mixes are shown in Table 2. These results correspond to samples extracted from the pavement. Table 3 shows the rheological properties, defined by the dynamic modulus and the fatigue law. Simple dynamic compression tests on the set of asphalt mixes determined the dynamic modulus, and samples from the binder layer gave the law of fatigue in stress-controlled four-point bending tests.

## PARAMETERS AND CRITERIA IN THE ANALYSIS

The comparison of the different sections was made by determining the number of applied loads that each one could withstand before failure.

The deterioration considered was

- Loss of surface evenness, and
- Loss of bearing capacity, assessed by
  - Cracking of the pavement surface and
  - Progression of the deflections.

The loss of surface evenness was understood to be the diversion of the surface throughout the test compared with that of the wearing course when the road was completed. The deterioration was evaluated directly by obtaining transverse profiles of the different stretches involved using a transverse profilograph.

The parameters taken into account in the analysis were rut depth in the center of the trafficked area, an absolute value or the difference between two consecutive evaluations (PRC or DRC in Spanish), and maximum rut depth of the trafficked area, an absolute value or the difference (PRM and DR). PRC and PRM were used to calculate the deterioration and its evolution during the test, and PRM and DR to evaluate the speed of this deterioration.

The measuring profiles were placed at equal intervals 1 m apart. The measurements taken were digitized to facilitate

**TABLE 1 Characteristics of Granular Materials**

| | ROUNDED GRANULAR MATERIAL | | CRUSHED ROCK | | SUBGRADE | |
|---|---|---|---|---|---|---|
| SIEVE SIZE - % PASSING | | | | | | |
| . 25 mm | 94,5 | | 100,0 | | 100,0 | |
| . 12,5 mm | 74,5 | | 67,0 | | 100,0 | |
| . 5 mm | 55,5 | | 51,5 | | 100,0 | |
| . 200 μm | 12,0 | | 13,5 | | 39,0 | |
| . 80 μm | 7,5 | | 9,5 | | 26,4 | |
| MODIFIED PROCTOR | | | | | | |
| . Density (t/m$^3$) | 2,18 | | 2,23 | | 1,95* | |
| . Optimum moisture (%) | 5,8 | | 7,0 | | 10,5 * | |
| RESILIENT MODULUS (MPa) ** $E = K_1 \left(\frac{I_1}{\sigma_0}\right)^{K_2}$ | $K_1$ (MPa) | $K_2$ | $K_1$ (MPa) | $K_2$ | $K_1$ (MPa) | $K_2$ |
| | 70,51 | 0,19 | 13,47 | 0,39 | 11,76 | 0,34 |

* Normal Proctor
** $I_1 = \sigma_1 + 2\sigma_3$
$\sigma_0 = 1$ KPa

**TABLE 2 Composition of the Mix (from Samples)**

| MIX TYPE | COMPOSITION | | | | | | | BITUMEN | |
|---|---|---|---|---|---|---|---|---|---|
| | BY WEIGHT | | | | BY VOLUME | | | | |
| | STONE (%) | SAND (%) | FILLER (%) | BITUMEN (%) | $V_A$ (%) | $V_b$ (%) | V (%) | PEN | $T_{RB}$ ºC |
| S-12 (wearing course) | 44,0 | 49,0 | 7,0 | 4,7 | 81,0 | 10,2 | 8,8 | 39 | 53 |
| S-20 (binder layer) | 47,0 | 46,5 | 6,5 | 4,5 | 82,9 | 9,9 | 7,2 | 37 | 53 |

**TABLE 3 Complex Moduli and Fatigue Characteristics of the Mix (Extracted Cores)**

| DYNAMIC MODULUS (MPa) (f = 10 Hz) | | | | FATIGUE LAW (f = 10 Hz) |
|---|---|---|---|---|
| T = 40ºC | T = 25ºC | T = 0ºC | T = -15ºC | |
| 1.100 | 6.000 | 13.000 | 18.000 | $\log \epsilon = -2{,}5 - 0{,}23 \log N$ |

computer processing and once the above-mentioned parameters were obtained, statistical analyses were made on individual sections and evolution curves were drawn up.

The following were considered to be criteria of generalized failure:

- PRC parameter:
  - Average PRC of the substretch ≥ 15 mm,
  - 20 percent or more of the profiles measured having a PRC ≥ 25 mm.

- PRM parameter:
  - Average PRM of the substretch ≥ 20 mm,
  - 20 percent or more of the profiles measured having a PRM ≥ 30 mm.

The appearance of surface cracking was another of the types of deterioration considered in the comparison of test sections. Data collection began with visual inspection. Each inspection surveyed the test sections and noted any cracks obvious to the naked eye when the operator was upright. Once the cracks were located they were marked with paint for subsequent identification and photographed with a superimposed metal grid to make measurement easier. The data were subsequently fed into the utilization of results program by means of digitization. This program then provided evaluation indexes automatically, calculating the representative parameters.

The analysis took into consideration the following parameters:

- Percentage length of cracked area (LF in Spanish),
- Percentage extent of cracked area (AF in Spanish), and
- Gravity of cracking, testing the cracked area with different weights as a function of the type of crack (GF in Spanish).

The failure criteria adopted were LF ≥ 50 percent, AF ≥ 40 percent, and GF ≥ 60.

Two pieces of equipment were used to evaluate the deflections: a KUAB double mass falling-weight deflectometer (FWD) with a maximum equivalent load of 50 kN and the Benkelman beam with a load of 130 kN, as standardized in Spain. The results will be presented in another paper (*1*).

## ANALYSIS OF RESULTS

### Surface Evenness

The data corresponding to the evolution of the rut depth (PRC) in average values per section according to the number of load cycles involved are shown in Figure 5. Table 4 gives corresponding deformation speeds (DRC/number of cycles).

After 1,000,000 test cycles, the order of the sections, from smaller to larger degree of deformation, was 1, 2, 3, 4, 5, and 6. The corresponding average rut depths were 12.4, 13.4, 18.4 (780,000 cycles), 26.6, 14.7 (500,000 cycles), and 15.8 (500,000

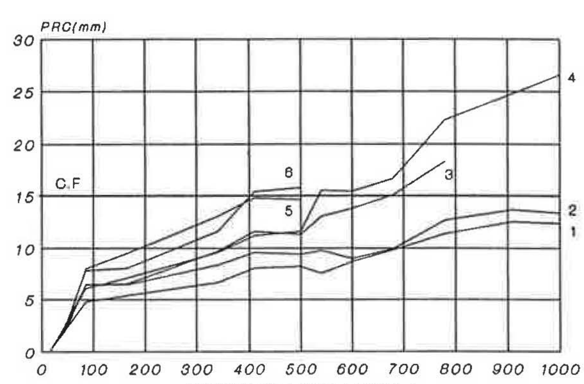


**FIGURE 5 Evolution of rut depth (PRC) with loading [CF = failure (maximum average PRC)].**

**TABLE 4 Evolution of Velocity of Formation of Rut Depth (VDRC, in $10^{-5}$ mm/cycle) with Loading (S = section; IC = number of loads × 1,000)**

| S \ IC | 15-50 | 50-85 | 85-165 | 165-340 | 340-410 | 410-500 |
|---|---|---|---|---|---|---|
| | VDRC | | | | | |
| 1 | 6,9 | 6,6 | 4,8 | 0,7 | 1,5 | 0,3 |
| 2 | 8,6 | 10,0 | 6,4 | 1,1 | 1,7 | 0,0 |
| 3 | 7,1 | 11,3 | 6,3 | 1,9 | 2,8 | 0,0 |
| 4 | 8,5 | 9,9 | 6,8 | 1,4 | 2,3 | 0,5 |
| 5 | 8,0 | 14,9 | 9,7 | 2,0 | 3,1 | 0,0 |
| 6 | 9,0 | 13,4 | 8,0 | 2,0 | 5,4 | 0,4 |

| S \ IC | 500-540 | 540-600 | 600-680 | 680-780 | 780-910 | 910-1000 |
|---|---|---|---|---|---|---|
| | VDRC | | | | | |
| 1 | 0,0 | 1,2 | 1,6 | 1,6 | 0,7 | 0,0 |
| 2 | 0,8 | 0,0 | 1,1 | 2,8 | 1,1 | 0,0 |
| 3 | 5,0 | 1,2 | 1,6 | 2,7 | - | - |
| 4 | 8,4 | 2,8 | 1,5 | 5,0 | 2,5 | 2,5 |
| 5 | - | - | - | - | - | - |
| 6 | - | - | - | - | - | - |

cycles). Sections 5, 6, and 3 contained deformations so great at certain points that they had to be repaired at 500,000, 600,000, and 780,000 cycles, respectively.

Sections 1 and 2 did not reach failure by any of the criteria established. Section 3 failed according to all the criteria at 700,000 cycles. Section 4 failed between 540,000 cycles and 700,000 cycles, depending on the criteria. Sections 5 and 6 failed at 550,000 and 500,000 cycles, respectively.

In the initial cycles (0 to 165,000), the deformation speed (Table 4) ranged between $6 \times 10^{-5}$ and $15 \times 10^{-5}$ mm/cycle. Between 165,000 and 1,000,000 cycles, deformation speeds were lower, with some exceptions. Standard Section 1 had a lower deformation speed than Section 4. The speed values were arranged in each group of sections according to the bearing capacity (thickness of asphalt) of the sections.

Figure 6 gives a month-by-month summary of the temperature and rainfall data recorded in the track during the test, and Figure 7 gives an example of the evolution of rut depth.

Three periods had very high temperatures: 50,000 to 85,000 cycles and 85,000 to 165,000 cycles, corresponding to the first summer, and 680,000 to 780,000 cycles, corresponding to the third summer (during the second summer the vehicle was stopped for modifications). The first two periods correspond to the greatest deformation speed, and the third one, although it has relatively low values, has greater speeds than the preceding and next cycles ($2.2 \times 10^{-5}$ to $4.8 \times 10^{-5}$ mm/cycle versus $1.3 \times 10^{-5}$ to $2.1 \times 10^{-5}$ mm/cycle). The large deformation that occurred in the initial cycles (with low and medium temperatures) is attributed mainly to postcompaction. Between 50,000 and 165,000 cycles postcompaction and high-temperature creep were added.

There were three rainy periods. The first two periods (15,000 to 50,000 cycles and 50,000 to 85,000 cycles) correspond to the first phase of large deformation speed, so it is impossible to separate the effect of the rain from the other factors. The third rainy period (500,000 to 540,000 cycles) was during the period that had low speeds, but again an increase in the speed with respect to the surrounding cycles is noted ($2 \times 10^{-5}$ to $10 \times 10^{-5}$ mm/cycle versus $0.4 \times 10^{-5}$ to $3.5 \times 10^{-5}$ mm/cycle). The influence of the rain is greater in pavements with cracks at the surface.

Table 5 presents data from cores of the zones in the trafficked area (taken as initial values) and from the trafficked area in the different sections. Comparing Table 5 with Figure 5 it can be seen that an important part of the deformation is due to settlement of the noncohesive materials. Asphalt mixes had an average initial 8 percent of voids (range between 7 and 11 percent) and underwent recompaction up to average

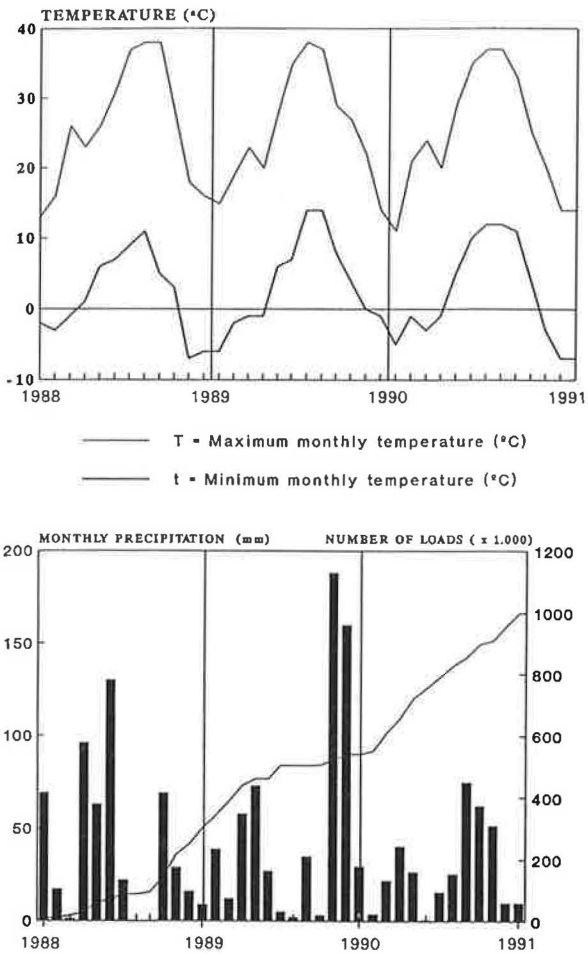


**FIGURE 6 Air temperature and precipitation history of test area.**

values of 4 percent (curves), 5 percent (Sections 1, 2, and 3 with 50 cm of granular layers), and 7 percent (Sections 4, 5, and 6 with 25 cm of granular layers). The end values did not depend on the initial voids. The corresponding percentages of the Marshall reference density were 101, 100, and 98 percent.

The information obtained with the PRC parameter (rut depth in the center of the two layers) was very similar to that obtained with the PRM parameter (maximum rut depth), with the maximum differences between the values of the parameters being 1 to 2 mm.

The failure criterion according to the maximum average value was more restrictive than the criterion by the measurements over a maximum value, and the PRC failure criteria were more restrictive than the PRM criteria.

### Surface Cracking

Figure 8 is a summary graph of the evolution of cracking, with the number of cycles for one part of a section, which is a direct output of the computer crack-processing program.

The wear initially manifested itself in the form of transverse cracks. This progressed to longitudinal cracks at the same time that the transverse cracks grew longer, forming the characteristic alligator crazing at the end.

As a function of the analysis of the samples taken from the pavement confirmed by a stress-strain analysis, the most likely hypothesis is that the cracking in the asphalt went through the following process:

- Loss of adhesion of the different asphalt layers involved,
- Failure through fatigue in the top layer,
- Failure of the intermediate layer, and
- Failure of the bottom layer.

For the sections tested, Figure 9 gives the deterioration curves formed by the LF parameter. As can be seen, the beginning of the deterioration did not appear continuously. Once the deterioration had started, the cracking speed was greater in damp and cold weather than in hot weather.

At the end of 1,000,000 cycles the sections were arranged in ascending order of cracking as follows: 1, 2, 4, 5, 3, and 6. This order pointed to a better durability of standard Section 1 than 4. In addition, standard Section 4 was more sensitive than Section 1 to cracking under traffic when reductions of thickness existed.

Within each standard section the sections were arranged in terms of surface cracking evolution according to the total thickness of the mix (the lower the thickness of the mix, the greater the cracking).

Section 1 had not reached failure point after 1,000,000 cycles and did not present any deterioration at the end of the test. In Section 2, 39 percent of the length and 22 percent of the surface area showed cracking. Section 3 had reached the failure point at 670,000 cycles (average of LF and AF). Section 4 reached failure at the end of the test, and Sections 5 and 6 after 500,000 cycles.

## GENERAL CONCLUSIONS

Section 322 of the Spanish Design Catalogue (Section 1 in the test), consisting of 15 cm of asphalt mix over 50 cm of granular material (25 cm of crushed rock and 25 cm of gravel), constitutes a better technical solution than Section 321 (Section 4 in the test), consisting of 18 cm of asphalt mix over 25 cm of crushed rock. This conclusion is based on two facts, brought out in the test:

- Section 322 withstood 1,000,000 load cycles without showing signs of any significant wear, whereas Section 321 showed 44.2 percent cracked length at the same point.
- Section 322 was less sensitive to any decreased thickness of the asphalt layers. The sections tested that had the same bottom layers as the preceding ones but 2 cm less pavement thickness showed very different behavior. With some of the criteria used, the one similar to Section 321 reached the failure point at 500,000 load cycles. The one analogous to Section 322 has not failed with 1,000,000 load applications even with 39 percent of cracked length.

Section 321 is not suitable for the working conditions seen in Spanish Design Standards 6.1 and 6.2-IC. This test revealed at least one situation, with a subgrade and climatic condition

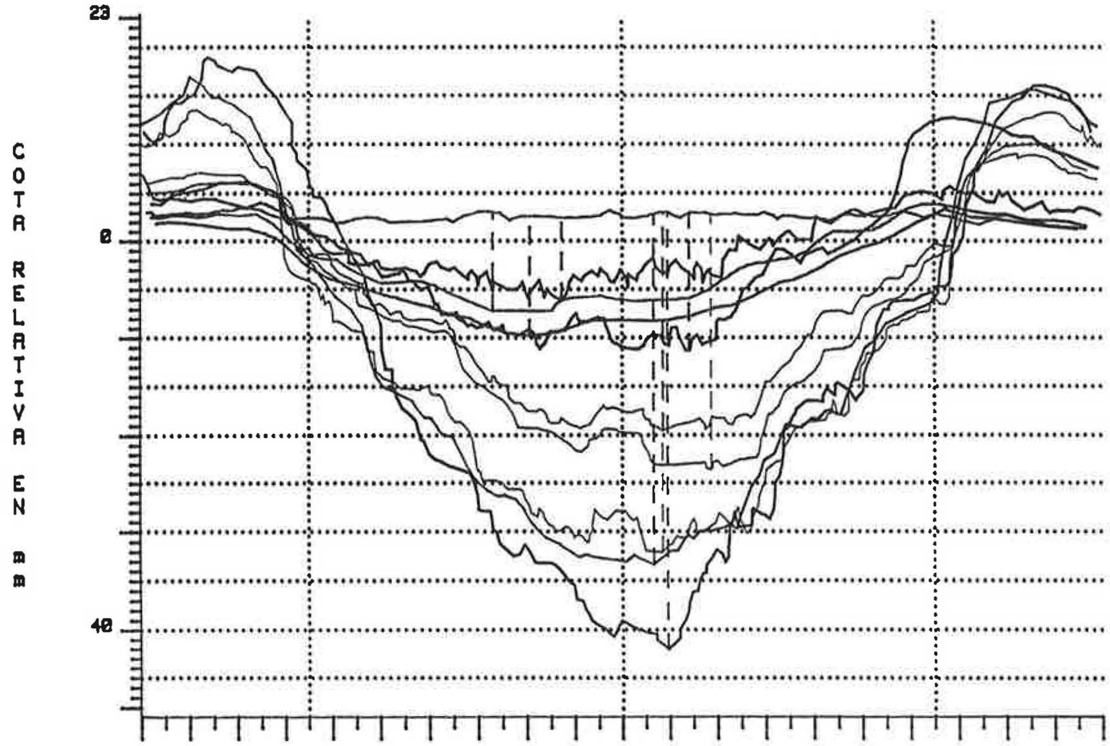


**FIGURE 7 Example of evolution of rut depth.**

very common for Spanish roads, in which the section did not fulfill the hypothesis of the catalogue. This section reached failure with 600,000 loads under rutting criteria and with 1,000,000 loads under cracking criteria. Consequently, in climatic conditions similar to those of Madrid (Figure 6), there is a great probability of its durability being between 12 and 15 years for a high level of traffic within the variation interval seen for its type in the road catalogue. For an average level of traffic within this interval, it would, however, meet the design conditions. Nevertheless, with 500,000 load applications, the cracked length in the test was over 15 percent. In practice this means that this structural section might require frequent maintenance and rehabilitation.

In the sections with 25 cm of granular base course layer placed directly on the subgrade, an asphalt mix thickness equal to or less than 18 cm can be considered critical since any small reductions in the thickness of the layers or in the quality of the materials cause rapid failure of the pavement.

**TABLE 5 Final Characteristics of the Asphalt Mix**

| SECTION | INITIAL THICKNESS (mm) | DECREASED THICKNESS (mm) |
|---|---|---|
| 1 | 155 | 6,1 |
| 2 | 125 | 7,0 |
| 3 | 105 | 6,6 |
| 4 | 160 | 6,0 |
| 5 | 145 | 13,0 |
| 6 | 120 | 10,0 |
| North Curve | 75 | 2,1 |
| South Curve | 75 | 5,1 |

In these sections exceeding the thickness by going from 15 to 18 cm meant a 20 percent increase in service life with regard to rutting or multiplied it by 2.5 with regard to cracking.

In the sections with 50 cm of granular layers, a pavement thickness equal to or less than 10 cm can be considered critical for the same reasons as those above. Because two of the sections did not reach failure at the end of the test, final figures cannot be given regarding the increase in service life when the 10-cm thickness is increased. What can be stated is that the service life of the section with a 12-cm thickness has proved to be at least 50 percent longer than that of the 10-cm-thick pavement.

The section composed of a 10.5-cm-thick asphalt layer and 50 cm of granular layers is more or less equivalent to one consisting of 16 cm of asphalt layer plus 25 cm of granular materials, since both withstood some 450,000 load cycles (12 years under average traffic and 8 with heavy traffic within the limits set). In other words, in this type of pavement, 1 cm of asphalt mix is the equivalent of 4 cm of gravel. These equivalences have been drawn up by composing the failure criteria of loss of evenness and cracking. If each type of failure is considered individually, the first section is slightly better than the second regarding evenness and also slightly worse in cracking, with equivalences of 1:3.3 and 1:4.5, respectively.

The relationship between the sections with noncritical thicknesses (Sections 1 and 2) and Sections 4, 5, and 6 is different, and it can be stated that an increase of 4 cm of granular material in a pavement creates a better service life than an increase of 1 cm in the asphalt layer.

As a consequence of the foregoing and for the reasons already given, it has been proposed, on the basis of the test results,

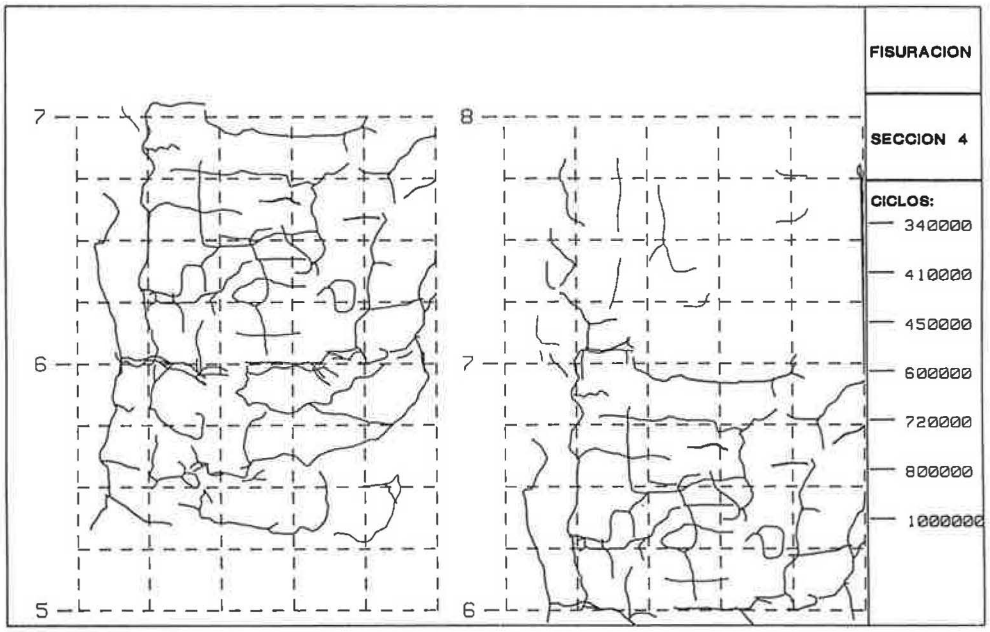


**FIGURE 8** Summary of cracking in Section 4.

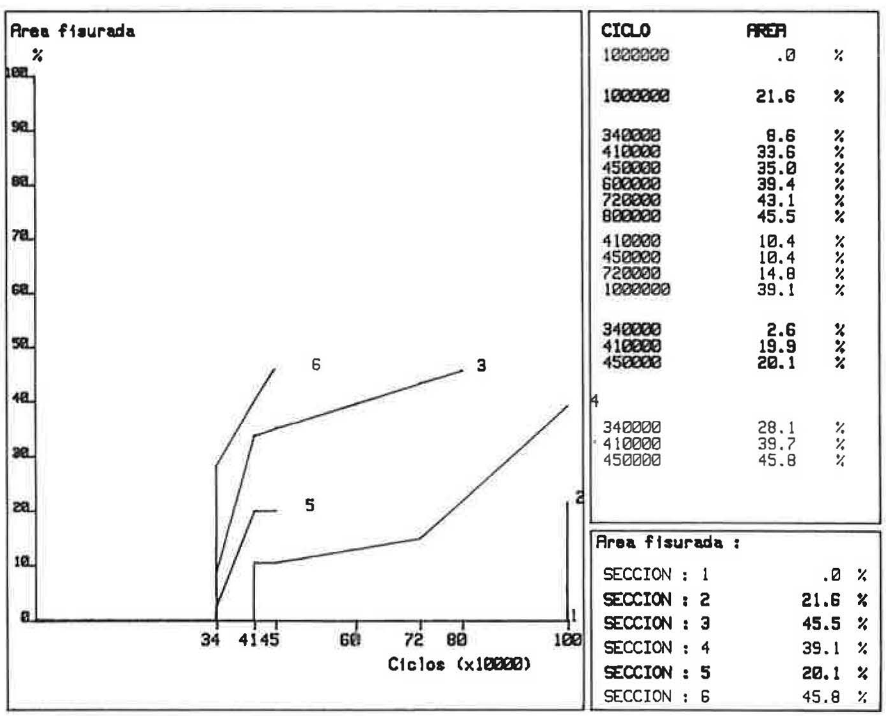


**FIGURE 9** Evolution of cracking (LF) with loading.

1. To eliminate Section 321 from the Spanish Design Catalogue, since its use could give rise to excessive maintenance costs or cause premature total failure, and
2. To keep Section 322—even though the asphalt thickness seems to be higher than necessary to bear the design traffic, a reduction in the thickness could lead to critical situations.

As regards the installation, the main conclusion reached has been that the installation for the testing of pavements created and built by CEDEX is perfectly valid for the purpose for which it was designed and has numerous advantages over the test tracks in existence.

## ACKNOWLEDGMENT

This research was sponsored by the General Directorate of Roads of the Ministry of Public Works and Transport, Spain.

## REFERENCE

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# Summary of Pavement Performance Tests Using the Accelerated Loading Facility, 1986–1990

RAMON BONAQUIST

FHWA has been conducting pavement performance tests using the Accelerated Loading Facility (ALF) for more than 5 years. Tests have been performed at the FHWA Pavement Testing Facility (PTF) during two research projects and at in-service sites in Montana and Wyoming. The first phase of pavement research at the PTF established operations and data collection procedures and assessed the rationality of performance data collected with the ALF. Data obtained during the PTF Phase 1 research showed that pavement performance under the ALF loading followed the general trends observed through monitoring of various test roads and in-service pavements. The in-service testing program demonstrated the mobility of the ALF, and experience with site preparation, traffic control, and site restoration was obtained. The rutting performance of an asphalt mixture specifically designed to resist rutting was evaluated in a Montana field test, which showed that the use of the antirutting mixture may delay the development of rutting, but that plastic flow, which has been identified as the major cause of severe rutting, was not eliminated. The PTF Phase 2 research, which compared the damage potential of conventional dual and wide-based single tires, featured the comparative testing capabilities of the ALF. In this experiment, the control was the dual-tire loading and the treatment was the wide-based single-tire loading. Findings to date show the wide-based single tire to be significantly more damaging to flexible pavements than conventional dual tires.

In 1984 FHWA began an accelerated pavement testing research program. The initial phase of this program was directed at the acquisition of a mobile, linear-tracking, accelerated loading device. Two mobile devices were operational at that time: the Australian Accelerated Loading Facility (ALF), and the South African Heavy Vehicle Simulator. In September 1984, FHWA entered into an agreement with the Department of Main Roads, New South Wales, Australia, to provide plans, specifications, and technical assistance for the construction of an ALF in the United States. Construction of the U.S. ALF began in July 1985, and the completed machine was delivered to the Turner-Fairbank Highway Research Center (TFHRC) in August 1986.

The initial phase of the FHWA accelerated pavement testing program also provided for the establishment of a permanent pavement testing laboratory, the Pavement Testing Facility (PTF), at TFHRC. The PTF was constructed in summer 1986 as an outdoor facility with eight test sections. It was subsequently expanded to 12 test sections during the first pavement reconstruction in 1989.

Federal Highway Administration, 6300 Georgetown Pike, McLean, Va. 22101.

The ALF has been in nearly continuous operation since its delivery to FHWA in 1986. From August 1986 through March 1989, the first phase of pavement research was conducted at the PTF. On completion of the Phase 1 research, a field testing program was conducted from April through December 1989. The field testing was performed in conjunction with the Western Association of State Highway and Transportation Officials (WASHTO), the states of Montana and Wyoming, and the U.S. Army Corps of Engineers. Test sections on I-90 near Columbus, Montana, and Sheridan, Wyoming, were tested, and a brief demonstration test was conducted for the Corps at the Waterways Experiment Station in Vicksburg, Mississippi. The ALF then returned to the TFHRC in January 1990 and the second phase of pavement research at the PTF began. For the Phase 2 research, the PTF was expanded to 12 test sections, and the ALF was modified to permit rapid movement between two adjacent test sections. This capability reduces environmental variations for comparative tests and is expected to play a major role in future PTF research programs.

## OBJECTIVE AND SCOPE

The accelerated pavement performance tests conducted with the ALF between August 1986 and December 1990 are summarized. During this period, tests were conducted on the 12 flexible pavements given in Table 1. For each of these tests, histories of rutting, cracking, roughness, and present serviceability index (PSI) were obtained as a function of the number of load applications. Environmental data including air and pavement temperatures and precipitation were collected during the tests to aid in the interpretation of the performance data. On completion of each test, postfailure analyses were conducted to document the condition of the pavement layers at the time of failure.

## ALF PAVEMENT TESTING MACHINE

The ALF pavement testing machine is a 100-ft-long structural frame containing a moving wheel assembly that travels 12 mph on rails attached to the frame and is in contact with the pavement for 38 ft. At the ends of the frame, the rails curve upward to permit gravity to accelerate, decelerate, and change direction of the wheel assembly. Loads are applied to the

**TABLE 1 Summary of Pavement Tests with the ALF 1986–1990**

| | | | Surface | | Base | | Loading | | |
|---|---|---|---|---|---|---|---|---|---|
| No. | Location | Dates | Type | Th. (in) | Type | Th. (in) | Load (lbs) | Tire Type | Pres. (psi) |
| 1 | PTF Phase 1, Lane 2, Section 3 | 01/08/87 - 06/04/87 | AC[a] | 7.0 | CAB[b] | 12.0 | 19,000 | Dual | 100 |
| 2 | PTF Phase 1, Lane 2, Section 2 | 06/18/87 - 11/30/87 | AC | 7.0 | CAB | 12.0 | 19,000 | Dual | 140 |
| 3 | PTF Phase 1, Lane 1, Section 2 | 12/14/87 - 02/18/88 | AC | 5.0 | CAB | 5.0 | 11,600 | Dual | 100 |
| 4 | PTF Phase 1, Lane 1, Section 4 | 03/01/88 - 03/08/88 | AC | 5.0 | CAB | 5.0 | 16,400 | Dual | 100 |
| 5 | PTF Phase 1, Lane 1, Section 1 | 03/24/88 - 04/04/88 | AC | 5.0 | CAB | 5.0 | 14,100 | Dual | 100 |
| 6 | PTF Phase 1, Lane 2, Section 1 | 04/29/88 - 12/03/88 | AC | 7.0 | CAB | 12.0 | 16,400 | Dual | 100 |
| 7 | PTF Phase 1, Lane 2, Section 4 | 01/09/89 - 02/23/89 | AC | 7.0 | CAB | 12.0 | 22,500 | Dual | 100 |
| 8 | I-90, Columbus, MT | 05/10/89 - 07/28/89 | AC | 9.8 | CAB | 20.4 | 19,000 | Dual | 100 |
| 9 | I-90, Sheridan, WY | 08/07/89 - 10/20/89 | AC | 4.8 | CTB[c] | 13.0 | 19,000 | Dual | 100 |
| 10 | PTF Phase 2, Lane 1, Section 4 | 01/10/90 - 04/30/90 | AC | 5.0 | CAB | 12.0 | 14,100 | Dual | 100 |
| 11 | PTF Phase 2, Lane 2, Section 1 | 09/20/90 - 01/18/91 | AC | 3.0 | CAB | 12.0 | 12,200 | Dual | 102 |
| 12 | PTF Phase 2, Lane 3, Section 1 | 09/20/90 - 01/18/91 | AC | 3.0 | CAB | 12.0 | 12,200 | Sgl | 102 |

[a] Asphalt Concrete
[b] Crushed Aggregate Base
[c] Cement Treated Base

pavement in one direction, and the loads can be distributed laterally to simulate traffic wander.

Figure 1 shows the trolley assembly used to apply loads to the test pavement. The wheel assembly can be detached from the trolley through a bolted connection at the elevation of the load cells (Figure 1a). The ALF has single- and dual-tire wheel assemblies that model one-half of a single axle. The loads applied to the pavement can be varied from 9,000 to 22,500 lb by adding or subtracting ballast weights. Thus, dual- or single-tire single axles with loads ranging from 18,000 to 45,000 lb can be simulated.

For all of the tests described in this paper, traffic wander was simulated using a normal distribution of lateral wheel positions with a standard deviation of 5.25 in. The normal distribution was truncated at 14.75 in., the maximum permissible lateral movement for the ALF. Figure 1b shows the geometry of the dual- and single-wheel assemblies used to apply the test loads. The centerline of the trolley moved through the lateral distribution, resulting in wheelpaths of 53 and 42 in. for the dual and single tires, respectively.

## PAVEMENT PERFORMANCE TESTING

### Data Collection

The objective of the pavement performance testing was to quantify the accumulation of structural and functional dis-

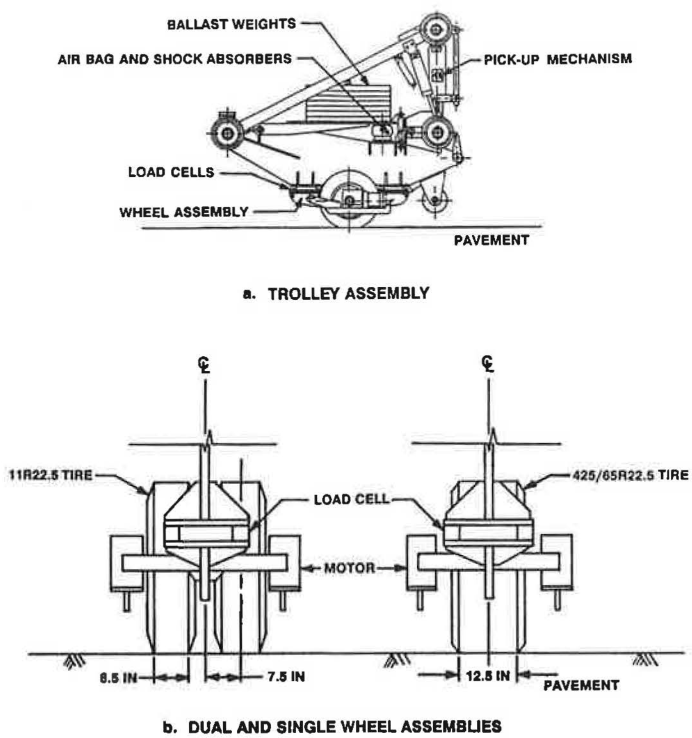


FIGURE 1 ALF loading.

tresses in the pavement test sections. Cracking and rutting data were obtained periodically during each of the accelerated load tests. The American Association of State Highway and Transportation Officials (AASHTO) serviceability concept was used to quantify the functional distress in terms of PSI. To supplement the performance data, postfailure investigations and materials testing were conducted on completion of the accelerated load testing. Since environmental control was not provided for any of the tests described in this paper, environmental conditions were closely monitored during each test to aid in the interpretation of the test data. Data collection equipment and procedures have been described in detail in previous reports (*1*,*2*).

### PTF Phase 1 Research Program

#### *Objectives*

The first phase of pavement research was conducted at the PTF from August 1986 through February 1989. The objectives of the Phase 1 research program were

1. To establish operating and data collection procedures for the PTF,
2. To assess the rationality of pavement response and performance data obtained with the ALF, and
3. To study pavement response and performance for a range of loads and tire pressures, with particular emphasis on the influence of tire pressure.

To accomplish these objectives, two types of tests, pavement response and accelerated loading, were conducted with the ALF. The response testing used the ALF's variable load capabilities and instrumentation installed in the test pavements. Response testing formed a major part of the tire pressure study conducted during the first phase of research. Details of the response testing and the tire pressure research have been presented elsewhere (*3*). The second type of test, accelerated loading, was used to collect pavement performance data as a function of the number of load repetitions. During accelerated loading, the ALF was controlled by a computer and operated 24 hr a day, 7 days a week. At typical productivity rates, approximately 5,500 load repetitions were applied to the test pavements daily.

#### *Test Sections and Material Properties*

For the first phase of research, the test sections at the PTF were constructed in two parallel lanes designated Lanes 1 and 2. Each lane was divided into four sections for a total of eight pavement test sections. The design cross sections consisted of a 2.0-in. asphalt concrete surface course and a 3.0-in. asphalt concrete binder course over a 5.0-in. crushed aggregate base for Lane 1, and a 2.0-in. surface course and 5.0-in. binder course over a 12.0-in. crushed aggregate base for Lane 2. Table 2 summarizes laboratory-determined material properties for each pavement layer.

**TABLE 2 PTF Material Properties, Phase 1**

Subgrade and Aggregate Base

| Property | Subgrade | Aggregate Base |
|---|---|---|
| Classification | A-4(0) | A-1-a |
| CBR | 7[a] | 100[b] |
| Average In-Situ Density | 108 pcf<br>96% AASHTO T-99 | 143.8 pcf<br>94% AASHTO T-180 |
| Average In-Situ Moisture | | |
| As Constructed | 10.0% | 3.2% |
| Post Failure | 17.0% | 5.5% |
| Resilient Modulus[c] | | |
| Reference 1 | | |
| $\sigma_d$ = 2 psi | 3400 $(\theta)^{0.52}$ | 5800 $(\theta)^{0.80}$ |
| $\sigma_d$ = 5 & 8 psi | 1700 $(\theta)^{0.52}$ | 13300 $(\theta)^{0.80}$ |
| Reference 4 | 3400 $(\theta)^{0.52}$ | 8830 $(\theta)^{0.23}$ |

Asphalt Concrete

| Property | Binder | Wearing |
|---|---|---|
| Asphalt Cement | AC-20 | AC-20 |
| Resilient Modulus, ksi[d] | | |
| 41 °F | 2224 | 1855 |
| 77 °F | 400 | 339 |
| 104 °F | 73 | 67 |
| Average Air Voids | 3.41% | 4.74% |
| Average Asphalt Content | 4.70% | 5.60% |

[a] Laboratory Soaked per ASTM D1883
[b] Estimated from Dynamic Cone Penetrometer data
[c] Models yield $M_r$ is psi for stresses in psi
$\sigma_d$ = deviatoric stress
$\theta$ = bulk stress
[d] From indirect tension tests

*Pavement Performance Results and Findings*

This section presents data from seven of the eight Phase 1 sections. The eighth section, Lane 1, Section 3, was used for initial shakedown testing of the ALF machine before the start of the pavement performance tests. The first phase of research included testing each lane with three load levels. For Lane 1, loads of 11,600, 14,100, and 16,400 lb were used, and loads of 16,400, 19,000 and 22,500 lb were used on Lane 2. All tests were conducted with dual 11R22.5 radial tires. The tire pressure was 100 psi for all sections except Lane 2, Section 2, which was tested at 140 psi as part of the tire pressure study.

Figures 2 and 3 summarize the loading history and the environmental and subgrade stiffness conditions for the Phase 1 tests. The subgrade moduli were estimated from falling weight deflectometer (FWD) data collected at untrafficked reference locations in Lanes 1 and 2 using the outer sensor deflection as described in AASHTO nondestructive testing (NDT) Method 2 (*4*). Because of the nonlinear behavior of the PTF subgrade soil, the estimated moduli may be somewhat higher than those occurring under the ALF loading. The nonlinear subgrade behavior also accounts for the difference in the estimated subgrade modulus between Lanes 1 and 2. The general trend in the modulus data shows a small decrease with time. This decrease was accompanied by a slight increase in subgrade moisture.

The results of the Phase 1 performance tests are summarized in Tables 3 and 4 for Lanes 1 and 2, respectively. Since

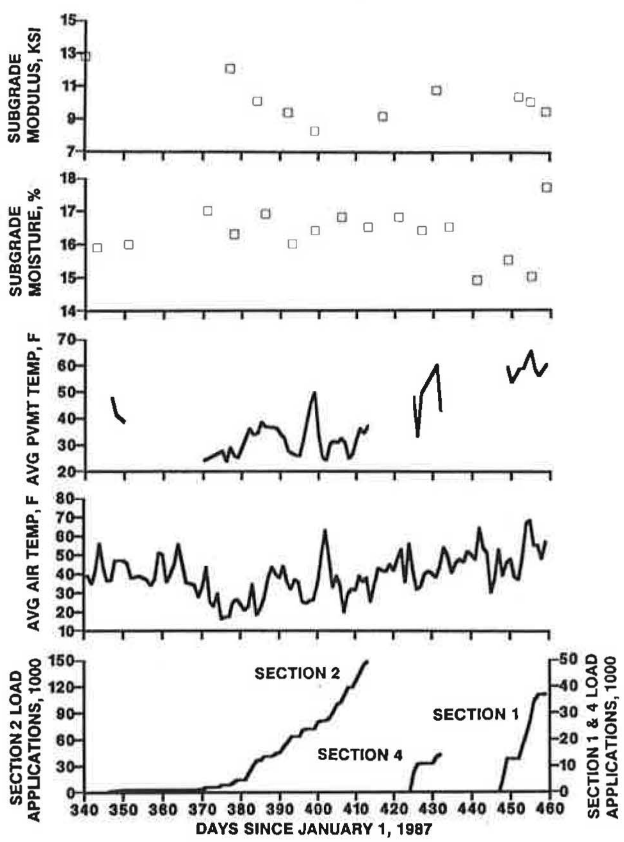


**FIGURE 2 Loading history and environmental conditions, PTF Phase 1, Lane 1.**

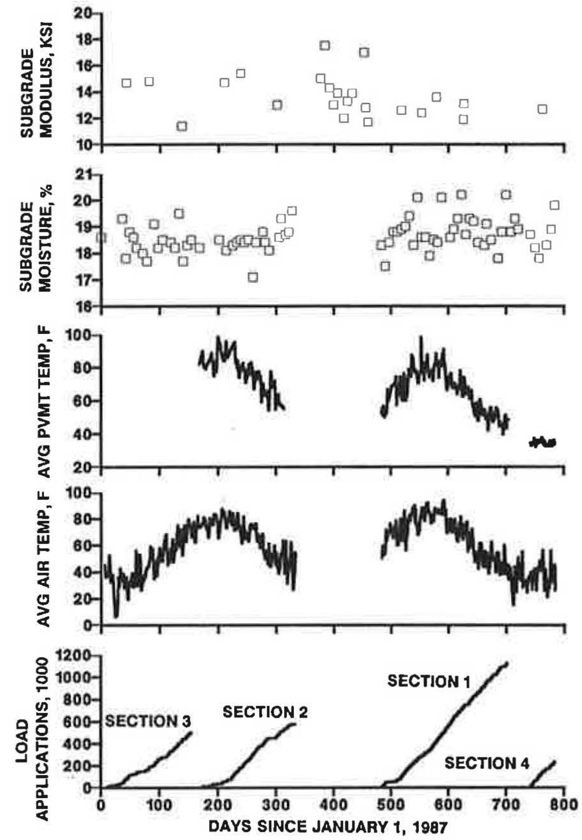


**FIGURE 3 Loading history and environmental conditions, PTF Phase 1, Lane 2.**

the ALF models half of a single axle, a pavement width of 6 ft was used in the computation of the percent cracking. The ALF wheelpath covers two-thirds of the 6-ft width; therefore, to obtain percent wheelpath cracking, the tabulated cracking data should be multiplied by 1.5.

Fatigue cracking was the predominant failure mode for the Phase 1 tests. Excessive rutting in the test sections did not develop until after the asphalt concrete was severely cracked. The test on Lane 2, Section 1, was cut short because of time constraints, but significant rutting and the onset of fatigue cracking were still observed in this test.

The pavement performance data obtained from the Phase 1 tests followed the general trends observed through monitoring of various test roads and in-service pavements. The effect of load on pavement damage was clearly evident. For tests conducted during warm weather, the rutting data showed rapid development of rutting, followed by a leveling-off period, followed by increased rutting after the initiation of fatigue cracks. As expected, the development of early rutting did not occur in the tests started during cold weather. The rapid development of fatigue cracking after crack initiation was clearly evident in all of the test sections. Fatigue cracks initiated transverse to the direction of travel of the ALF. After repeated load applications, longitudinal and additional transverse cracks appeared, resulting in the block or alligator cracking typical of fatigue failure. Finally, the loss of serviceability observed in the ALF tests was similar to that observed during the AASHO Road Test.

**TABLE 3 Performance Summary: PTF Phase 1 Research, Lane 1**

| Test Section | Date | No. of Passes | Slope Variance ($10^{-6}$) | Avg Rut Depth (in) | Avg Cracking (%)[a] | PSI Loss |
|---|---|---|---|---|---|---|
| Lane 1, Section 1 | | | | | | |
| | 3/24/88 | 0 | 4.35 | 0.00 | 0 | 0.00 |
| | 3/28/88 | 12529 | 7.62 | 0.15 | 0 | 0.43 |
| | 3/30/88 | 21909 | 24.47 | 0.27 | 1 | 1.42 |
| | 3/31/88 | 26875 | 36.82 | 0.34 | 5 | 1.85 |
| | 4/ 1/88 | 34443 | 43.48 | 0.44 | 5 | 2.09 |
| | 4/ 4/88 | 37033 | 57.97 | 0.57 | 39 | 2.64 |
| Lane 1, Section 2 | | | | | | |
| | 12/14/87 | 0 | 1.78 | 0.00 | 0 | 0.00 |
| | 1/11/88 | 8481 | 1.78 | 0.03 | 0 | 0.00 |
| | 1/22/88 | 40875 | 2.03 | 0.05 | 0 | 0.07 |
| | 1/27/88 | 51782 | 3.43 | 0.10 | 4 | 0.46 |
| | 2/ 1/88 | 70685 | 12.02 | 0.15 | 21 | 1.45 |
| | 2/ 9/88 | 91013 | 23.20 | 0.16 | 21 | 1.97 |
| | 2/15/88 | 126485 | 41.00 | 0.21 | 29 | 2.48 |
| | 2/18/88 | 147696 | 79.28 | 0.28 | 36 | 3.08 |
| Lane 1, Section 4 | | | | | | |
| | 3/ 1/88 | 0 | 2.73 | 0.00 | 0 | 0.00 |
| | 3/ 3/88 | 10876 | 14.72 | 0.40 | 9 | 1.60 |
| | 3/ 8/88 | 13301 | 39.69 | 0.74 | 35 | 2.93 |
| | 3/ 9/88 | 14240 | 50.39 | 1.06 | 39 | 3.93 |

[a] Based on total pavement width. For wheelpath cracking multiply by 1.5.

Figures 4 and 5 present layer profiles obtained from the Phase 1 postfailure testing. For the Lane 2 tests, no rutting was observed in the subgrade; therefore, only the profiles of the pavement surface and the surface of the crushed aggregate base course are shown in Figure 5. For most of the Phase 1 tests, the majority of the rutting occurred in the crushed aggregate base layer. Rutting in the subgrade was only observed for tests using heavy loads on the thin pavement structure of Lane 1. The permanent deformation in the asphalt layer was small for all tests. Even Sections 1 and 2 of Lane 2, which were trafficked primarily during hot weather, exhibited less than 0.38 in. of rutting in the asphalt concrete.

## Montana and Wyoming Field Tests

### *Objectives*

In 1989 a field testing program for the ALF was conducted in conjunction with WASHTO and the states of Montana and Wyoming. The objectives of this program were to document the benefits, costs, and difficulties associated with using the ALF to test in-service pavements and to evaluate the measures being taken in the WASHTO states to combat premature rutting in asphalt pavements (*5*).

### *Test Sections and Material Properties*

The outer wheelpath of two pavement sections on I-90 was tested as part of this study. The first section, near Columbus, Montana, was a pavement rehabilitated in 1986 in accordance with the WASHTO antirutting guidelines (*6*). At the time of rehabilitation, the pavement cross section consisted of 6.0 in. of asphalt concrete over a 20.4-in. granular base. The asphalt concrete, produced in accordance with the WASHTO antirutting guidelines, included an original 4.2-in. layer constructed in 1972 and a 1.8-in. overlay placed in 1978. The rehabilitation consisted of milling the 1978 overlay and placing a 1.8-in. inlay in the driving lane followed by a 3.0-in. overlay and 0.75-in. open graded friction course over both lanes.

The second test section, near Sheridan, Wyoming, was a cement-treated base section constructed in 1983. The pavement consisted of a 0.75-in. friction course and a 4.0-in. asphalt concrete surface course over a 13.0-in. cement-treated base. Table 5 summarizes laboratory-determined material properties for both the Montana and Wyoming test sections.

### *Pavement Performance Results and Findings*

Figure 6 presents the loading and temperature histories for the Montana and Wyoming field tests. Both sections were tested with a 19,000-lb load on dual 11R22.5 radial tires inflated to 100 psi. Table 6 summarizes the performance data collected during the two field tests.

Both pavements had been trafficked but were in excellent condition before testing with the ALF. An estimated 750,000 18-kip equivalent single-axle loads (ESALs) had been applied to the Montana site since the 1986 rehabilitation. This traffic resulted in a 0.19-in. rut before the start of trafficking with the ALF. On completion of the test, the rut depth increased to 0.71 in. Observations during postfailure sampling and testing indicated that all the rutting occurred in the asphalt layer. Using the AASHTO 38,000-lb single-axle load equivalency of 11.2 (for a terminal serviceability of 3.0), the Montana section exceeded its design axle loading before the development of a 0.75-in. rut, the critical value established by the WASHTO committee for this study. Although the design life was exceeded, transverse surface profiles obtained during testing

**TABLE 4 Performance Summary: PTF Phase 1 Research, Phase 2**

| Test Section | Date | No. of Passes | Slope Variance ($10^{-6}$) | Avg Rut Depth (in) | Avg Cracking (%)[a] | PSI Loss |
|---|---|---|---|---|---|---|
| Lane 2, Section 1 | | | | | | |
| | 4/29/88 | 0 | 8.59 | 0.00 | 0 | 0.00 |
| | 6/ 8/88 | 137714 | 6.10 | 0.10 | 0 | 0.00 |
| | 6/17/88 | 190419 | 6.10 | 0.12 | 0 | 0.00 |
| | 6/23/88 | 223427 | 7.76 | 0.16 | 0 | 0.00 |
| | 7/ 7/88 | 288022 | 8.53 | 0.18 | 0 | 0.00 |
| | 7/14/88 | 316082 | 9.20 | 0.20 | 0 | 0.05 |
| | 7/21/88 | 339821 | 11.46 | 0.22 | 0 | 0.21 |
| | 7/27/88 | 377470 | 20.52 | 0.24 | 0 | 0.67 |
| | 8/ 3/88 | 425977 | 25.25 | 0.29 | 0 | 0.83 |
| | 8/17/88 | 519440 | 25.55 | 0.32 | 0 | 0.84 |
| | 8/24/88 | 566618 | 26.96 | 0.34 | 0 | 0.88 |
| | 8/31/88 | 623192 | 28.08 | 0.35 | 0 | 0.92 |
| | 9/21/88 | 749838 | 29.50 | 0.34 | 0 | 0.96 |
| | 10/12/88 | 849598 | 30.61 | 0.34 | 0 | 0.98 |
| | 10/26/88 | 934859 | 33.70 | 0.35 | 0 | 1.06 |
| | 11/ 3/88 | 976131 | 35.39 | 0.34 | 0 | 1.10 |
| | 11/18/88 | 1061098 | 42.54 | 0.34 | 2 | 1.25 |
| | 11/23/88 | 1091880 | 53.46 | 0.34 | 3 | 1.44 |
| | 12/ 3/88 | 1125385 | 61.55 | 0.34 | 4 | 1.55 |
| Lane 2, Section 2 | | | | | | |
| | 6/18/87 | 0 | 12.84 | 0.00 | 0 | 0.00 |
| | 7/21/87 | 37292 | 12.24 | 0.23 | 0 | 0.04 |
| | 8/18/87 | 130082 | 11.10 | 0.57 | 0 | 0.34 |
| | 10/15/87 | 450895 | 45.31 | 0.63 | 3 | 1.60 |
| | 11/30/87 | 578142 | 82.89 | 0.89 | 25 | 2.75 |
| Lane 2, Section 3 | | | | | | |
| | 1/ 5/87 | 0 | 10.03 | 0.00 | 0 | 0.00 |
| | 2/12/87 | 77475 | 6.96 | 0.05 | 0 | 0.00 |
| | 3/12/87 | 146896 | 10.84 | 0.08 | 0 | 0.07 |
| | 4/21/87 | 276949 | 21.56 | 0.30 | 0 | 0.75 |
| | 5/18/87 | 416812 | 41.54 | 0.76 | 2 | 2.03 |
| | 6/ 4/87 | 502662 | --[b] | -- | 9 | -- |
| Lane 2, Section 4 | | | | | | |
| | 1/ 9/89 | 0 | 2.57 | 0.00 | 0 | 0.00 |
| | 1/18/89 | 44407 | 3.49 | 0.06 | 0 | 0.19 |
| | 1/25/89 | 81202 | 3.08 | 0.08 | 0 | 0.12 |
| | 2/ 1/89 | 131121 | 7.82 | 0.10 | 0 | 0.76 |
| | 2/ 8/89 | 164873 | 31.63 | 0.12 | 0 | 1.87 |
| | 2/15/89 | 186777 | 120.64 | 0.14 | 3 | 3.01 |
| | 2/27/89 | 233622 | 201.80 | 0.33 | 19 | 3.63 |

[a] Based on total pavement width. For wheelpath cracking multiply by 1.5.
[b] Data not collected

indicated the occurrence of plastic flow in the asphalt. Figure 7 shows the individual dual-tire tracks; plastic heave outside the wheelpath, accounting for almost one-half of the total rut depth, was clearly evident in the Montana pavement.

For the Wyoming site, an estimated 650,000 18-kip ESALs had been applied since construction in 1983. The test section exhibited typical cement-treated-base shrinkage cracks, which had been sealed as part of the normal maintenance program for this pavement. No rutting was evident at the start of the ALF test. The Wyoming test section was not significantly affected by the ALF traffic. Only a minor amount of rutting and a slight increase in roughness were detected.

## PTF Phase 2 Research Program

### *Objectives*

While the ALF field tests were being conducted, the test pavements at the PTF were reconstructed for the Phase 2 research program. During this reconstruction, the PTF was expanded to include three parallel test lanes with a total of 12 pavement test sections.

Cold weather construction problems plagued the Lane 1 construction, rendering three of the four test sections untestable. The fourth section was tested in conjunction with the Organization for Economic Cooperation and Development (OECD) First OECD Road Common Experiment (FORCE) project (*7*). Funding and technical expertise for the FORCE project were obtained from 14 countries. The project included the conduct of a common accelerated load test at the circular pavement testing facility in Nantes, France, as well as several cross-checking accelerated tests at facilities in participating countries. Lane 1, Section 4, of the Phase 2 test sections served as the U.S. cross test.

The eight test sections in Lanes 2 and 3 were devoted to a comparative study of the damaging effect of dual and wide-based single tires. The study, which will be completed in the summer of 1992, included two complementary experiments: pavement response and pavement performance. The objec-

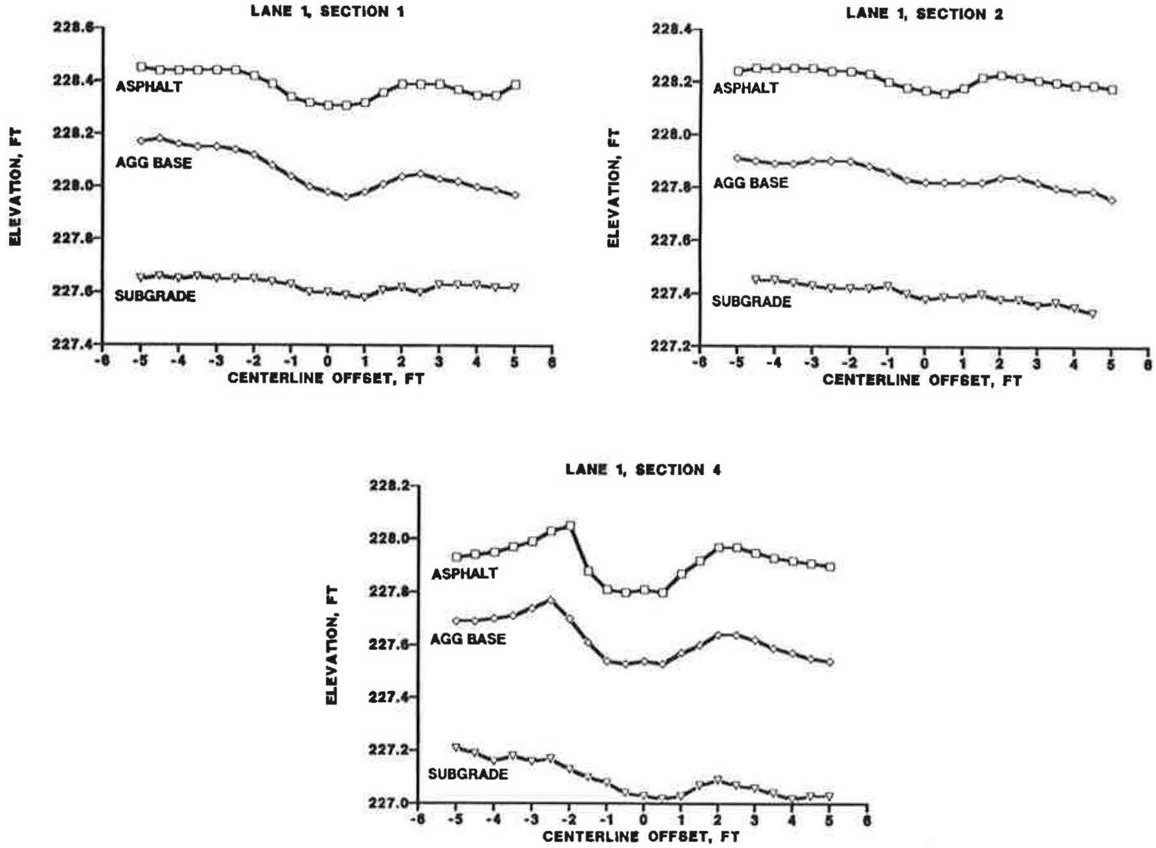


FIGURE 4 Postfailure profiles, PTF Phase 1, Lane 1.

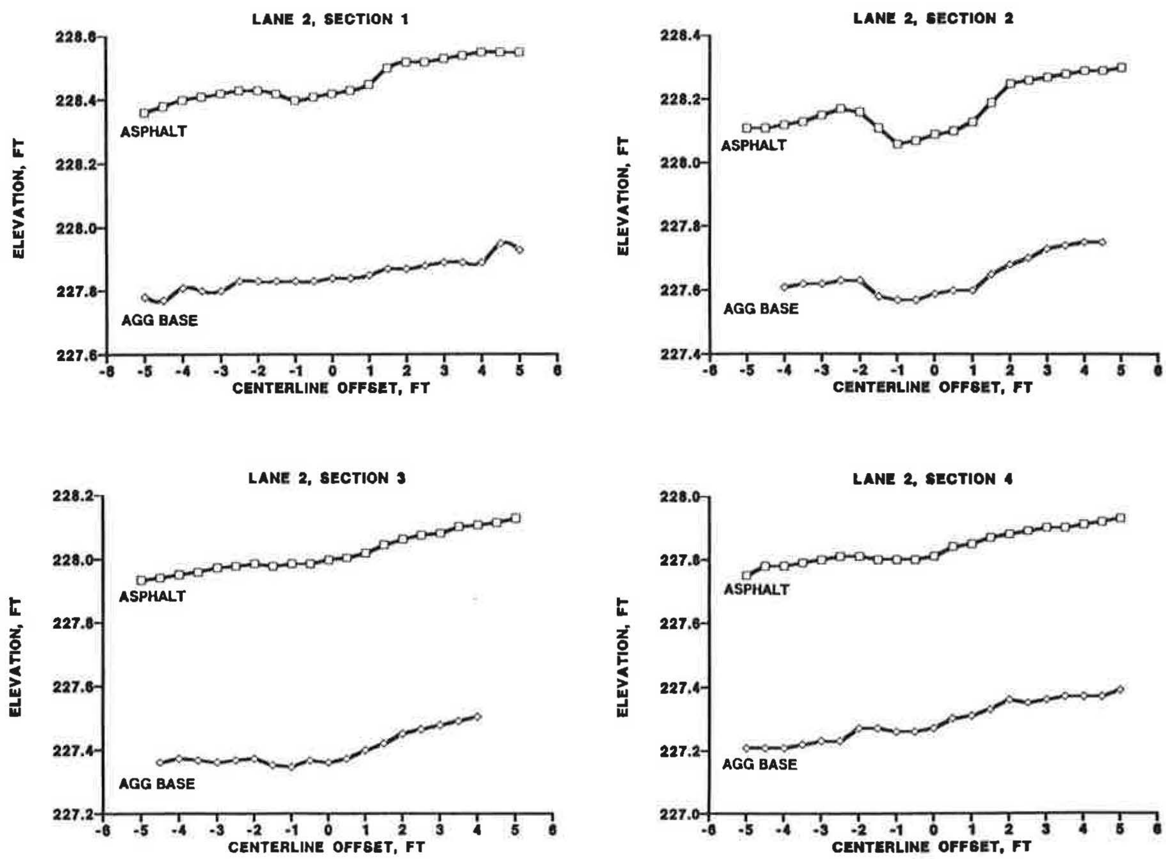


FIGURE 5 Postfailure profiles, PTF Phase 1, Lane 2.

**TABLE 5 Material Properties, Montana and Wyoming Field Tests**

Subgrade and Base

| Property | Montana Subgrade | Montana Base | Wyoming Subgrade | Wyoming Base |
|---|---|---|---|---|
| Classification | A-4(1) | A-1-a | A-6(10) | Cement Treated |
| Strength | R Value 32 | R Value 80 | R Value 5 | 3421 psi |
| Average In-Situ Density | 118.7 pcf<br>98% AASHTO T-99 | 133.9 pcf<br>96% AASHTO T-180 | 113.3 pcf<br>95 % AASHTO T-99 | NT[a] |
| Average In-Situ Moisture | | | | |
| Before ALF Testing | 11.1% | 4.4% | 16.6% | NT |
| After ALF Testing | 13.4% | 4.3% | 16.0% | NT |
| Resilient Modulus[b] | NT | NT | $10327\sigma_d^{-.36}$ | NT |

Asphalt Concrete

| Property | Montana Anitrut | Montana Original | Wyoming Surface |
|---|---|---|---|
| Asphalt Cement | 85/100 Pen | 120/150 Pen | AC-20 |
| Resilient Modulus, ksi[c] | | | |
| 55 °F | 1560 | 1710 | NT |
| 73 °F | 479 | 695 | NT |
| 90 °F | 51 | 169 | NT |
| Average Air Voids | | | |
| Before ALF Testing | 2.75% | 3.10% | 5.65% |
| After ALF Testing | 2.10% | 3.30% | 4.68% |
| Average Asphalt Content | 5.70% | 5.90% | 5.39% |

[a] Not Tested
[b] Models yield $M_r$ is psi for stresses in psi
$\sigma_d$ = deviatoric stress
[c] From indirect tension tests

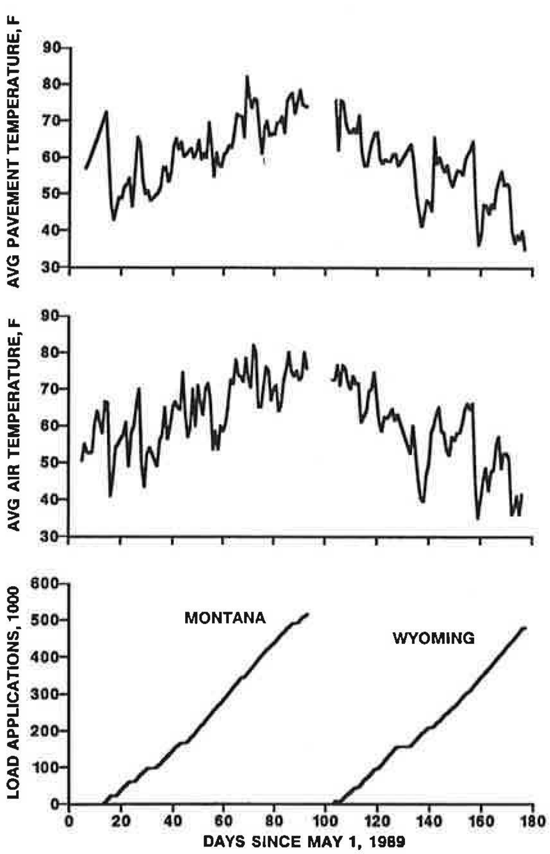


**FIGURE 6 Loading history and environmental conditions, Montana and Wyoming field tests.**

tive of the response experiment was to assess the relative damage potential of wide-based single tires through an analysis of pavement responses (strains and deflections). The objective of the performance experiment was to assess the relative damage potential of wide-based single tires through a comparative analysis of pavement performance. Only the comparative performance tests on Lane 2, Section 1, and Lane 3, Section 1, were completed during the time frame covered by this paper.

### *Test Sections and Material Properties*

The design cross section for Lane 1, Section 4, the OECD cross-test section, consisted of a 2.0-in. asphalt concrete surface course and a 3.0-in. asphalt concrete binder course over a 12.0-in. crushed aggregate base. The design cross section for Section 1 of Lanes 2 and 3, which was used in the dual-versus single-tire performance test, was a 1.0-in. asphalt concrete surface course and a 2.5-in. binder course over a 12.0-in. crushed aggregate base. The aggregate base and the subgrade soil were the same as those in the PTF Phase 1 tests. Table 7 presents material properties for the asphalt concrete used in the Phase 2 test sections.

### *Pavement Performance Results and Findings*

Figure 8 presents the loading and temperature histories for the Phase 2 tests. The OECD section was tested with a load of 14,100 lb on dual 11R22.5 radial tires inflated to 100 psi.

TABLE 6 Performance Summary: Montana and Wyoming Field Tests

| Test Section | Date | No. of Passes | Slope Variance ($10^{-6}$) | Avg Rut Depth (in) | Avg Cracking (%)[a] | PSI Loss |
|---|---|---|---|---|---|---|
| I-90, Montana | | | | | | |
| | 5/10/89 | 0 | 4.70 | 0.19 | 0 | 0.00 |
| | 5/16/89 | 31750 | 4.80 | 0.19 | 0 | 0.01 |
| | 5/22/89 | 63338 | 5.30 | 0.19 | 0 | 0.08 |
| | 5/31/89 | 103118 | 5.50 | 0.19 | 0 | 0.11 |
| | 6/ 6/89 | 148133 | 5.50 | 0.19 | 0 | 0.11 |
| | 6/13/89 | 181353 | 6.10 | 0.28 | 0 | 0.24 |
| | 6/20/89 | 238786 | 6.20 | 0.33 | 0 | 0.29 |
| | 6/27/89 | 293464 | 6.00 | 0.33 | 0 | 0.27 |
| | 7/ 5/89 | 354227 | 6.60 | 0.45 | 0 | 0.47 |
| | 7/11/89 | 403003 | 7.10 | 0.59 | 0 | 0.72 |
| | 7/18/89 | 454496 | 8.50 | 0.62 | 0 | 0.91 |
| | 7/28/89 | 514480 | 8.50 | 0.71 | 0 | 1.08 |
| I-90, Wyoming | | | | | | |
| | 8/ 7/89 | 0 | 2.60 | 0.00 | 0 | 0.00 |
| | 8/15/89 | 34667 | 2.70 | 0.00 | 0 | 0.02 |
| | 8/22/89 | 79392 | 2.60 | 0.00 | 0 | 0.00 |
| | 8/29/89 | 127599 | 3.00 | 0.00 | 0 | 0.09 |
| | 9/ 6/89 | 154946 | 2.60 | 0.00 | 0 | 0.00 |
| | 9/12/89 | 197022 | 2.90 | 0.00 | 0 | 0.07 |
| | 9/20/89 | 243822 | 2.70 | 0.00 | 0 | 0.02 |
| | 9/27/89 | 292295 | 2.80 | 0.00 | 0 | 0.04 |
| | 10/ 3/89 | 340159 | 2.80 | 0.00 | 0 | 0.04 |
| | 10/10/89 | 395750 | 3.20 | 0.00 | 0 | 0.13 |
| | 10/20/89 | 477737 | 3.50 | 0.05 | 0 | 0.19 |

[a] Based on total pavement width. For wheelpath cracking multiply by 1.5.

The dual- and single-tire tests used a load of 12,000 lb and 102-psi tire pressure. Lane 2, Section 1, was tested with dual 11R22.5 tires, and Lane 3, Section 1, was tested with a 425/65R22.5 wide-based single tire. During the conduct of these tests, the ALF was alternated between the two test sections to minimize environmental effects. On the basis of tests at the time of construction and during the postfailure investigation, there was no significant difference in subgrade or base course moisture between the dual- and single-tire performance sections.

The results of the Phase 2 performance tests are summarized in Table 8. The test for each of these sections was stopped

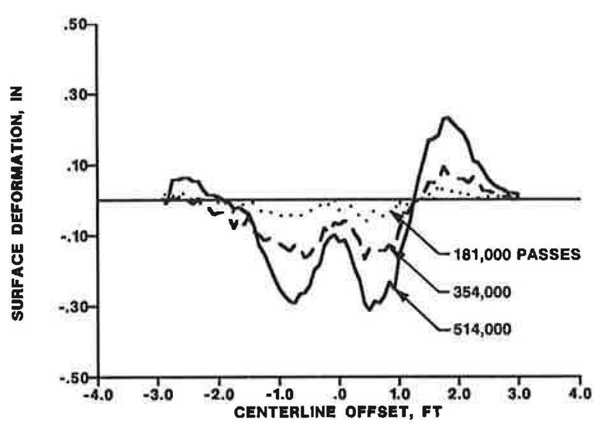


FIGURE 7 Typical transverse profile, Montana test section.

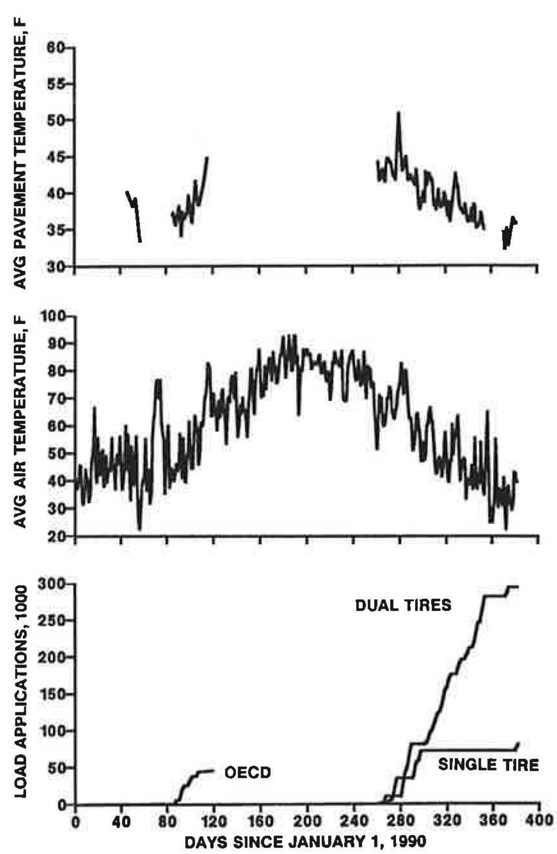


FIGURE 8 Loading history and environmental conditions, PTF Phase 2.

**TABLE 7 PTF Material Properties, Phase 2**

Asphalt Concrete

| Property | Section 1, Lanes 2&3 Binder | Section 1, Lanes 2&3 Wearing | Section 4, Lane 1 Binder | Section 4, Lane 1 Wearing |
|---|---|---|---|---|
| Asphalt Cement | AC-20 | AC-20 | AC-20 | AC-20 |
| Resilient Modulus, ksi[a] | | | | |
| 41 °F | 1770 | 1267 | 1413 | 1152 |
| 77 °F | 267 | 199 | 244 | 239 |
| 104 °F | 37 | 36 | 40 | 40 |
| Average Air Voids | 2.60% | 7.04% | 4.20% | 8.90% |
| Average Asphalt Content | 4.72% | 5.69% | 4.40% | 5.60% |

[a] From indirect tension tests

upon initiation of fatigue failure in the test section. Fatigue failure was identified by a sharp increase in the observed cracking in the test section. Fatigue failure of the OECD section occurred after 34,000 load applications. A comparison of the performance data for this section with that obtained on a pavement of similar cross section at the Nantes facility will be performed as part of the FORCE project. For the dual- versus single-tire performance test, the wide-based single tire proved to be significantly more damaging to thin asphalt pavements. The fatigue life of the single-tire section was approximately one-fourth that for the dual-tire section, and the rutting under the single tire was approximately twice that for the comparable dual tires.

Figure 9 presents layer profiles obtained from the Phase 2 postfailure testing. No rutting in the subgrade was observed for these tests; therefore, only the profiles for the pavement surface and the surface of the crushed aggregate base course are shown. Like the PTF Phase 1 tests, the majority of the rutting occurred in the crushed aggregate base layer. The permanent deformation in the asphalt layer was small for all three tests.

**TABLE 8 Performance Summary: PTF Phase 2 Research**

| Test Section | Date | No. of Passes | Slope Variance ($10^{-6}$) | Avg Rut Depth (in) | Avg Cracking (%)[a] | PSI Loss |
|---|---|---|---|---|---|---|
| Lane 1, Section 4 | | | | | | |
| | 1/10/90 | 0 | 4.69 | 0.00 | 0 | 0.00 |
| | 2/21/90 | 104 | 4.65 | 0.03 | 0 | 0.00 |
| | 2/21/90 | 113 | 4.51 | 0.04 | 0 | 0.00 |
| | 2/27/90 | 231 | 4.40 | 0.06 | 0 | 0.00 |
| | 3/29/90 | 1103 | 4.68 | 0.12 | 0 | 0.02 |
| | 3/30/90 | 5103 | 4.64 | 0.13 | 0 | 0.02 |
| | 4/ 3/90 | 10103 | 4.72 | 0.19 | 0 | 0.06 |
| | 4/ 4/90 | 19103 | 4.77 | 0.29 | 0 | 0.17 |
| | 4/ 5/90 | 24103 | 5.22 | 0.35 | 1 | 0.32 |
| | 4/11/90 | 34103 | 6.96 | 0.49 | 13 | 0.72 |
| | 4/18/90 | 44103 | 10.44 | 0.61 | 28 | 1.27 |
| | 4/26/90 | 45140 | 10.91 | 0.62 | 28 | 1.32 |
| Lane 2, Section 1 | | | | | | |
| | 9/18/90 | 0 | 6.90 | 0.00 | 0 | 0.00 |
| | 9/21/90 | 1000 | 9.56 | 0.09 | 0 | 0.25 |
| | 9/25/90 | 5000 | 10.62 | 0.10 | 0 | 0.33 |
| | 9/27/90 | 10000 | 11.60 | 0.11 | 0 | 0.40 |
| | 10/12/90 | 35000 | 16.17 | 0.16 | 0 | 0.68 |
| | 10/18/90 | 80000 | 21.10 | 0.19 | 0 | 0.90 |
| | 11/ 9/90 | 119846 | 21.85 | 0.21 | 0 | 0.94 |
| | 11/16/90 | 153188 | 24.50 | 0.21 | 0 | 1.03 |
| | 11/30/90 | 193897 | 27.42 | 0.24 | 0 | 1.14 |
| | 12/14/90 | 236256 | 54.19 | 0.24 | 4 | 1.76 |
| | 1/ 7/90 | 280000 | 80.20 | 0.34 | 9 | 2.19 |
| | 1/15/90 | 293017 | 150.68 | 0.41 | 10 | 2.78 |
| Lane 3, Section 1 | | | | | | |
| | 9/27/90 | 0 | 4.64 | 0.00 | 0 | 0.00 |
| | 9/27/90 | 1000 | 5.62 | 0.10 | 0 | 0.15 |
| | 10/ 1/90 | 5000 | 11.73 | 0.18 | 0 | 0.72 |
| | 10/ 2/90 | 10000 | 11.85 | 0.20 | 0 | 0.74 |
| | 10/ 8/90 | 34120 | 24.67 | 0.27 | 0 | 1.36 |
| | 10/29/90 | 71044 | 39.46 | 0.36 | 0 | 1.83 |
| | 1/21/90 | 79456 | 102.73 | 0.51 | 9 | 2.87 |

[a] Based on total pavement width. For wheelpath cracking multiply by 1.5.

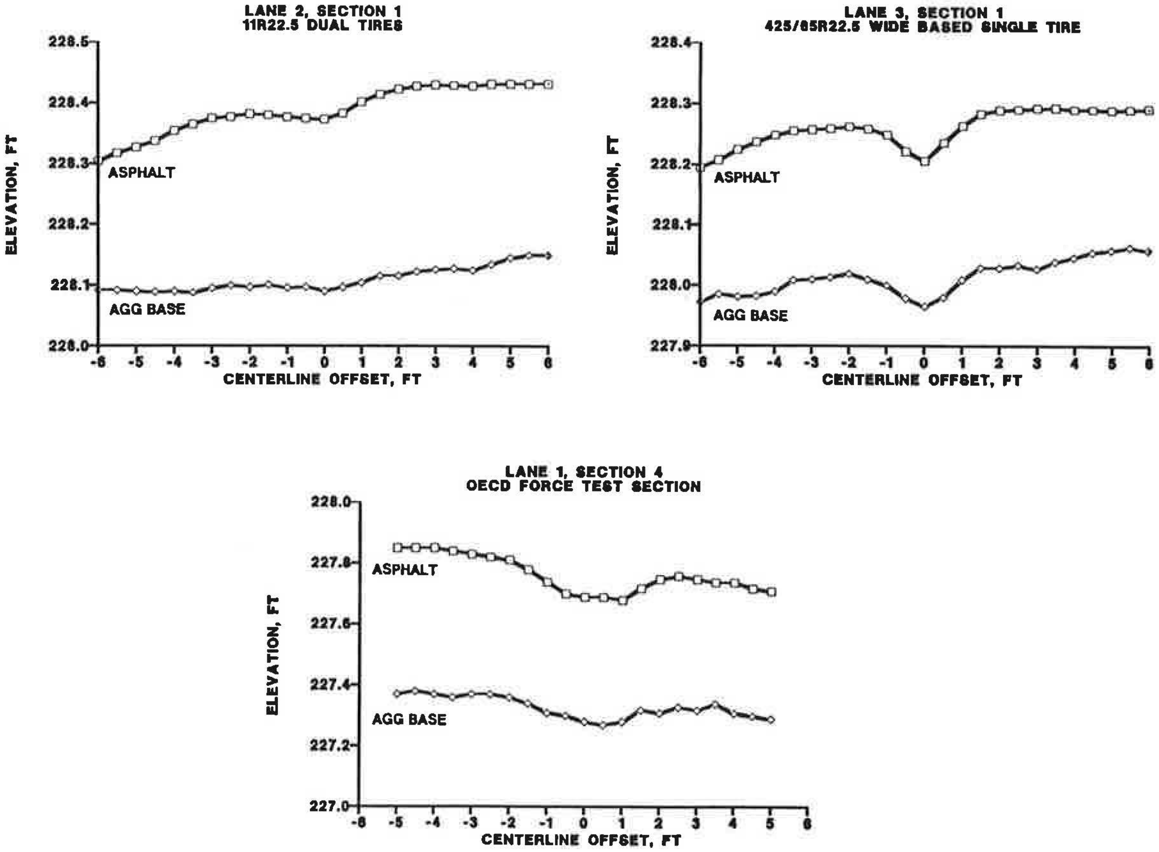


**FIGURE 9 Postfailure profiles, PTF Phase 2.**

## SUMMARY AND CONCLUSIONS

FHWA has been conducting accelerated pavement performance tests using the ALF machine since August 1986. Tests have been conducted at the PTF as well as at field sites in Montana and Wyoming. During the PTF Phase 1 research, much of the early work was directed at the establishment of operating and data collection procedures for the ALF and the PTF. Once these procedures were established, several accelerated pavement performance tests were conducted to assess the rationality of data obtained with the ALF. These tests showed that performance data obtained with the ALF followed the general trends observed through monitoring of various test roads and in-service pavements. The ALF and the instrumented PTF pavements were also used to study the effect of increased tire pressure.

The field testing program demonstrated the mobility of the ALF and experience with site preparation, traffic control, and site restoration was obtained. This experience was very important, because some of the future research studies identified for the ALF, particularly those associated with maintenance and rehabilitation, require the use of in-service pavements. The Montana field test was an excellent example of the benefits of accelerated pavement testing. During this 3-month test, the rutting performance of an asphalt mixture specifically designed to resist rutting was evaluated. This evaluation showed that the use of the antirutting mixture may delay the development of rutting, but that plastic flow, which has been identified as the major cause of severe rutting, was not eliminated. Additional mixture design research combined with verification through accelerated testing is definitely needed.

The current Phase 2 research at the PTF demonstrated the comparative testing capabilities of the ALF. In this experiment, the control was the dual-tire loading and the treatment was the wide-based single-tire loading. Findings to date show the wide-based single tire to be significantly more damaging to flexible pavements than conventional dual tires. During the PTF Phase 2 research, techniques were developed to eliminate environmental effects during comparative testing. Future ALF research will emphasize the use of comparative testing in the evaluation of pavement materials and pavement sections.

The above conclusions are based on the stated objectives of the various research programs. An additional use of the performance data presented in this paper is the validation of mechanistic-empirical pavement analysis concepts. Each test represents a valid observation of pavement performance for the materials, loading conditions, and environment encountered. These data should be useful in a variety of validation and model development efforts.

## ACKNOWLEDGMENT

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