Evolution of Long-Lasting Asphalt Pavement Design Methodology: A Perspective

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INTRODUCTION

The purpose of this paper is to provide a perspective on the evolution of long-lasting asphalt pavement design methodology. At the outset, however, it must be stated that this perspective is based on the author's view of developments in this area beginning in 1953, at which time his involvement in research in the areas of *flexible pavement design* and *asphalt technology* began.

As will be seen, these developments are the result of many engineers in the international community freely sharing their ideas and research through technical meetings, individual contacts made possible by such meetings, published technical papers, reports, and correspondence.

Perspectives such as this are necessarily limited by the experiences and contacts of the preparer. In the author's case, the perspective is influenced significantly by the International Conferences on Asphalt Pavement Design (starting at the University of Michigan in 1962) and the contacts made through these conferences, particularly with individuals from other parts of the United States, Europe, South Africa, and Australia. Moreover, this discussion has been significantly influenced by the engineers and researchers with whom he has had the privilege to work in California.

The ensuing discussion is based on two premises:

1. *Long lasting asphalt pavements can be designed using the developments in mechanistic-empirical (analytically-based) design over the past 40-plus years.*
2. *While long lasting asphalt pavements can be designed, careful attention to good construction practices is required to insure the anticipated design performance.*

Some of the key developments during the past 40-plus years which are briefly summarized include:

1. Mechanistic analyses,
2. Materials characterization,
3. Mechanistic-Empirical (M-E) pavement design methodologies,
4. Non-destructive pavement testing,
5. Accelerated pavement testing,
6. Improved construction practices, and
7. Pavement management.

Figure 1 contains a timeline of some of the key developments in each of these areas.

To conclude this discussion, some observations on *education* and *training* are presented since the two premises stated above are based on the fact that well educated engineers and skilled technicians are required to insure that long lasting asphalt pavements can be designed and constructed.

BACKGROUND/FOUNDATIONS

As will be seen subsequently, 1962 serves as the starting point, in the author's view, for the design of long-lasting asphalt pavements using analytically-based methodology as we know it today. However, there were a number of developments prior to this time which had a significant influence. Some of these will be briefly described in this section while others will be included in the specific key developments as they are discussed.

A significant contribution was the work of the U.S. Army Corps of Engineers (USACE) during World War II to develop a pavement design procedure for airfield pavements, initially for military applications. This procedure made use of the California Bearing Ratio (CBR) procedure developed by O.J. Porter for the California Highway Department (*1*). The USACE, in addition to modifying the CBR test procedure to meet their needs, also modified the thickness design curve

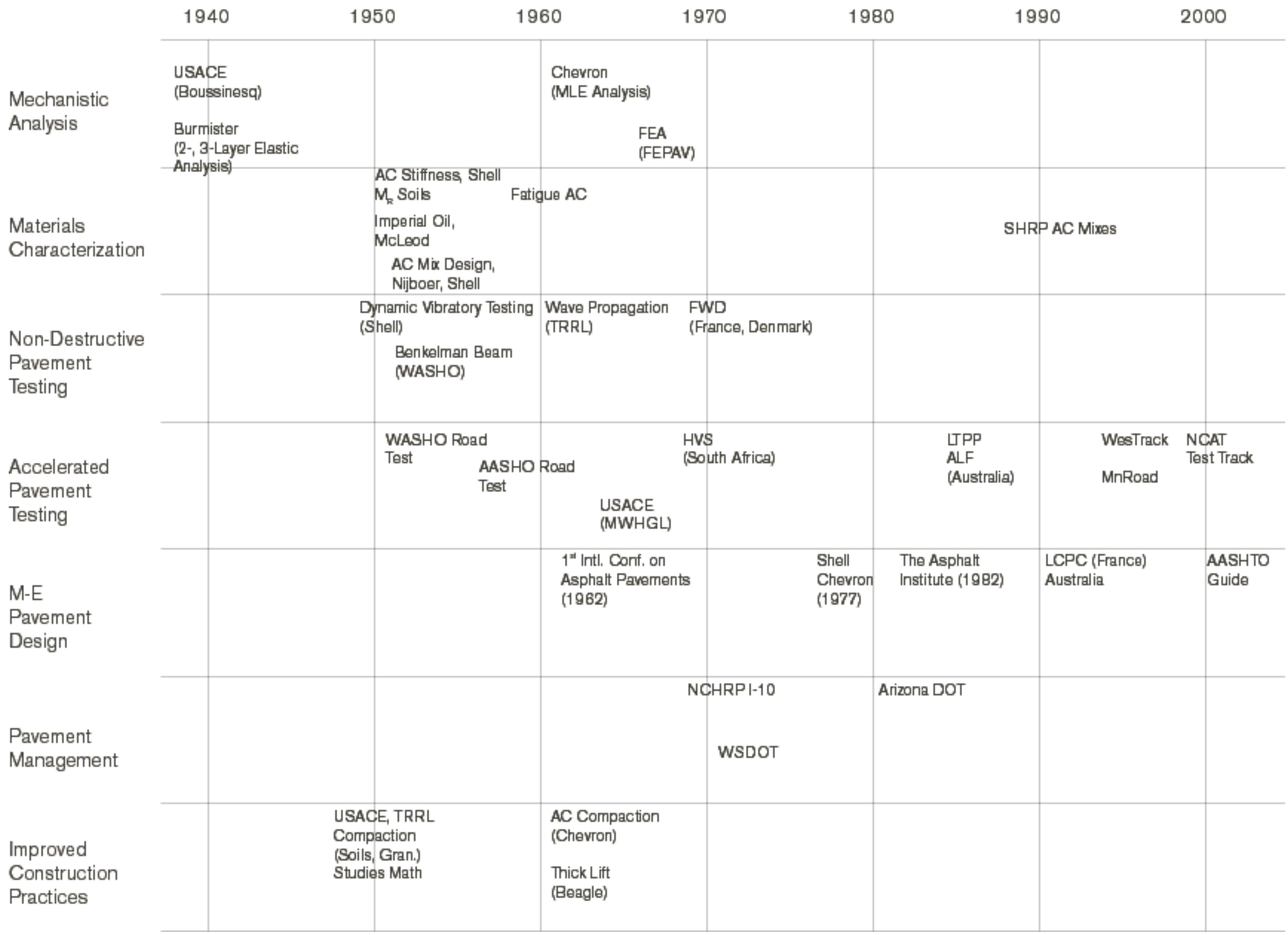

Figure 1. Timeline for some key developments in design and construction of long lasting asphalt pavements.

developed for highway loading to accommodate a range in aircraft wheel loads using elastic theory (Boussinesq) (*2*). Initially the aircraft operated on single wheel gears. In 1945, the B-29 was introduced with dual-wheel gears. This required additional considerations of the use of the Boussinesq solution and resulted in the introduction of the Equivalent Single Wheel Load (ESWL)[1] concept (*3*) which is still in use today. Two of the key people in this development were W. Turnbull and R. Ahlvin (*4*) (Figure 2). Reference (*4*) contains useful solutions to the Boussinesq solid developed by the USACE, which were used extensively until the advent of the electronic computer in the 1960's. A key feature of the USACE studies was the use of accelerated pavement testing to validate and modify thicknesses arrived at for different aircraft load and gear configurations by the analytical procedure using the Boussinesq analysis (*2*).

R.G. Alvin

W. Turnbull

Figure 2. R.G. Alvin and W. Turnbull.

[1] Aircraft gear loads are expressed in terms of an equivalent single wheel load (EWSL) defined as the single wheel load which yields the same maximum deflection at a given depth as a multiple wheel load; the contact area of this ESWL is equal to the contact area of one of the wheels of the multiple wheel assembly. As will be seen subsequently, this definition is different than the equivalent single axle load (ESAL) used for highway pavement design.

Test roads have been used in the United States since at least 1921 with recorded reference to the Bates Test Road.[2] Two key developments in this area were the Western Association of State Highway Officials (WASHO) Road Test in Malad, Idaho in 1951 (*5*) and the American Association of State Highway Officials (AASHO) Road Test in the period 1958-60 in Ottawa, Illinois (*6*).

At the WASHO Road Test, A.C. Benkelman introduced the Benkelman Beam which permitted pavement deflections to be measured under slow moving wheel loads (*5*). This tool 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 overlay pavement design. In this test road, the importance of thicker sections of asphalt concrete [4 in. (100 mm) versus 2 in. (50 mm)] to improve pavement performance was also demonstrated.

The AASHO Road Test[3] (*6*) sparked a renewed interest in improved pavement design and provided the impetus for the development of many current analytically-based design procedures. Under the excellent leadership of W.N. Carey, Jr. (Figure 3), 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 these data. Performance predictions by the new analytically-based procedures could be compared with actual field performance; reasonable comparisons confirmed the “engineering reasonableness” of the methodologies.

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[2] Discussed in Older, C. “The Bates Experimental Road,” and Goldbeck, A.T. Highway Researches and What the Results Indicate,” papers in *Proceedings of the American Road Builders’ Association,* 1922.
[3] Resulted from the 1956 Interstate Highway Act; its cost of $29 million would correspond to the cost of the Strategic Highway Research Program 30 years later, 1988-1993, which was $150 million.

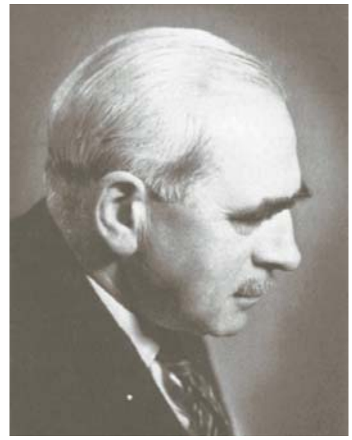
Figure 3. W.N. Carey, Jr.

During this period the Road Research Laboratory (RRL) [now the Transportation Research Laboratory (TRL)] of the United Kingdom installed test sections in a number of their major roadways to study the longer term performance of pavements under actual traffic loading in specific environments. One such experiment was the Alconbury Hill motorway reported by Croney et al. (e.g., Reference *7*) (Figure 4). At the time, Sir William Glanville (Figure 4) was the Laboratory Director and pavement research received considerable emphasis under his direction.

Another key development during this period in the pavement analysis area was the presentation of solutions by D. Burmister in the 1940's for the response of two-and three-layer elastic systems to representative loading conditions (*8*). While 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.

The work of F.N. Hveem (Figure 5) and his staff in California also contributed significantly to the development of M.E. design. Pavement deflections were under investigation

D. Croney



Sir William Glanville

Figure 4. D. Croney and Sir William Glanville.

Figure 5. F.N. Hveem.

by Hveem for a number of years prior to the WASHO Road Test using a GE travel gauge. Publication of his research in 1955 (*9*) provided a strong link between pavement deflections, truck loading, and fatigue failures in the asphalt-bound portion of pavement sections; Figure 6 illustrates some of his results.

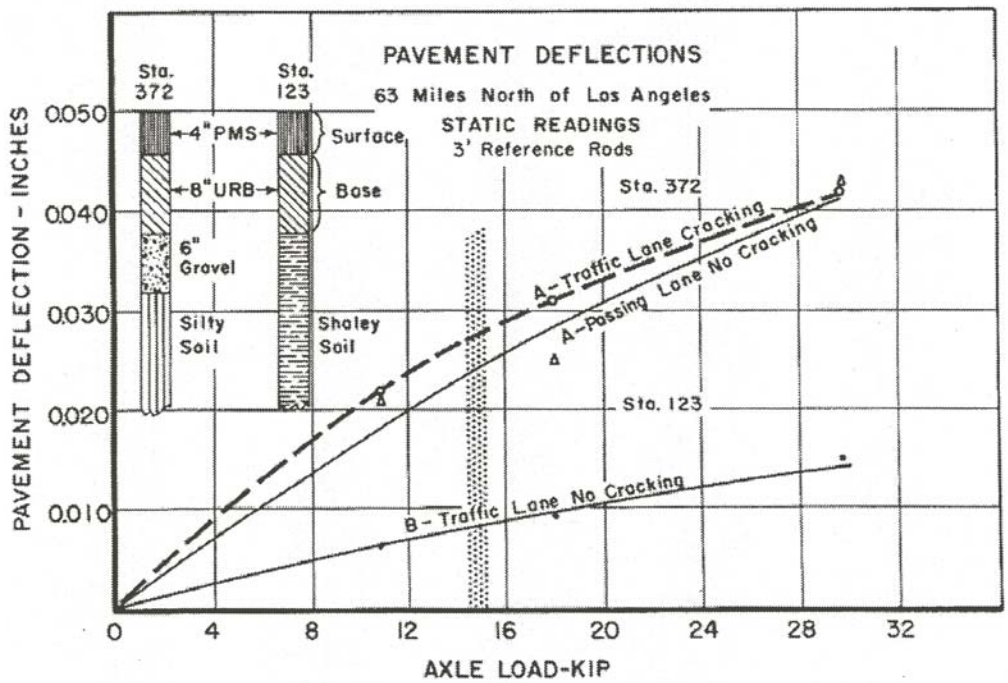


Figure 6. Pavement deflections, cracked and uncracked sections.(*9*)

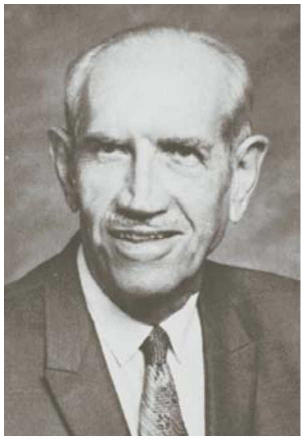
Hveem's work had a most significant impact on the development of procedures to predict fatigue cracking using analytically-based methodologies. In addition to relating pavement deflection to HMA fatigue, Hveem also introduced the concept of equivalent wheel loads (EWL) (*10*), the forerunner of equivalent single axle loads (ESALs), and the concept of layer equivalency with the use of the gravel equivalent factor (*11*).

As noted in the introduction, the first International Conference on Asphalt Pavements (termed The International Conference on the Structural Design of Asphalt Pavements) was convened at the University of Michigan in August 1962. The conference was conceived by F.N. Finn (who had been the Asphalt Institute's representative at the Test Road) (Figure 7) to provide a technical venue for discussion of the results of the AASHO Road Test, as well as for worldwide developments in asphalt pavement design. J.E. Buchanan (Figure 8), the President of

Figure 7. F.N. Finn.

Figure 8. J.E. Buchanan.

the Asphalt Institute, strongly supported the concept. Because of its long time association with asphalt pavements, the University of Michigan (UM) at Ann Arbor was selected as the conference site. W.S. Housel and W.K. Parr (Figure 9) of the UM Civil Engineering Department, working with Asphalt Institute representatives and with key U.S. and international members of the asphalt paving community, developed a very successful conference. While many of the elements for mechanistic empirical (analytically-based) pavement design were being worked on prior to the Conference, the framework for this approach "gelled" there; particularly the efforts of the Shell investigators (*15, 16*), the Asphalt Institute (*14*), and the RRL of the UK (*13*).



W.S. Housel

W.K. Parr

Figure 9. W.S. Housel and W.K. Parr.

A few of the key early U.S. and international participants are shown in Figure 10 (E.J. Yoder, W. Goetz, K. Wester, E. Nakkel, P. Rigden, and J. Kirk). The bound volumes of these Conferences, in addition to containing the technical papers, moderator reports, and discussions, include listings of the various committees which have contributed to the continued success of these conferences.

The sections to follow provide some discussion of each of the key developments listed earlier.

MECHANISTIC ANALYSIS

The use of multi-layered analysis to represent pavement response, although developed by Burmister (Figure 11) in the 1940s (*8*), did not receive widespread attention until the First International Conference on the Structural Design of Asphalt Pavements in 1962. While some agencies utilized solutions for two- and three-layered elastic solids in their design methodologies [e.g., the U.S. Navy (*12*)], the use of these solutions was both limited and cumbersome.


W.H. Goetz


E.J. Yoder

U.S. participants


K. Wester
Netherlands


P. Rigden,
South Africa


E. Nakkel
West Germany


J. Kirk,
Denmark

International participants

Figure 10. Key U.S. and International participants, 1962-1967.

Figure 11. D.M. Burmister.

At the 1962 Conference, however, important contributions were made by Whiffin & Lister (*13*), Skok & Finn (*14*), Peattie (*15*), and Dormon (*16*). Both Whiffin & Lister and Skok & Finn illustrated how layered elastic analysis could be used to analyze pavement distress. Peattie & Dorman presented several 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 multi-layered elastic solids also were presented at the 1962 Conference. Additional related work was published in 1967 at the Second International Conference. These general solutions, coupled with rapidly advancing computer technology, fostered the development of the current generation of multi-layer elastic and viscoelastic computer programs. Table 1 contains a listing of *some* of the most commonly used programs. The ELSYM program, developed at the University of California, Berkeley by G. Ahlborn (*17*) and widely used, directly benefited from the 1962 and 1967 Conference papers.[4]

Computer solutions for layered systems in which the properties of each of the layers could be represented as linear viscoelastic materials were subsequently introduced; two available solutions, VESYS (*23*) and VEROAD (*24*), are listed in Table 1.

In the late 1960s, finite-element analyses to represent pavement response were developed by a number of researchers [e.g., Duncan, et al. (*29*)]. Increasingly, the finite-element method has been used to model pavement response, particularly to describe the nonlinear response characteristics of pavement materials. Examples of this approach include ILLIPAVE (*25*) and FENLAP (*26*).

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[4] Although the work of the CHEVRON researchers never appeared in the published literature, it is important to recognize their significant contribution since they presented the first computer solution for a five-layer system (CHEV5L) in 1963 (*18*).

Table 1 Summary of Some Available Computer-Based Analytical Solutions for Asphalt Concrete Pavements.

Program	Theoretical Basis	Number of Layers (max)	Number of Loads (max)	Program Source	Remarks
BISAR (*19*)	MLE	5	10	Shell International	The program BISTRO was a forerunner of this program
ELSYM (*17*)	MLW	5	10	FHWA (UCB)	Widely used MLE analysis program
PDMAP (PSAD) (*20*)	MLW	5	2	NCHRP Project I-10	Includes provisions for iteration to reflect non-linear response in untreated aggregate layers
JULEA (*21*)	MLE	5	4+	USACE WES	Used in Program LEDFAA
CIRCLY (*22*)	MLE	5+	100	MINCAD, Australia	Includes provisions for horizontal loads and frictionless as well as full-friction interfaces
VESYS (*23*)	MLE or MLVE	5	2	FHWA	Can be operated using elastic or viscoelastic materials response
VEROAD (*24*)	MLVE	15 (resulting in half-space)		Delft Technical University	Viscoelastic response in shear; elastic response for volume change
ILLIPAVE (*25*)	FE		1	University of Illinois	
FENLAP (*26*)	FE		1	University of Nottingham	Specifically developed to accommodate non-linear resilient materials properties
SAPSI-M (*27, 28*)	Layered, damped elastic medium	N layers resting on elastic half-space or rigid base	Multiple	Michigan State University/ University of California Berkeley	Complex response method of transient analysis—continuum solution in horizontal direction and finite element solution in vertical direction

MLE—multilayer elastic
MLVE—multilayer viscoelastic
FE—finite element

Recently, solutions for the dynamic analysis of asphalt concrete pavements under moving, fluctuating loads have been developed. The SAPSI-M program (*28*), Table 1, is one such example. In this program moving loads are modeled as a series of pulses with durations equal to the time required for a load to pass a specific location.

In terms of current analytically-based pavement design procedures, layered elastic analysis is the primary method for defining pavement response to load. Use of the finite element methodology has had limited application to date [e.g., ILLIPAVE (*25*)] possibly because of computational time constraints. Hence, it has been used primarily in special applications. However, improvements in both computer capabilities and in formulating finite element representations should allow it to become an integral part of routine pavement analysis of asphalt pavements in the future. [It should be noted that finite element analysis is an integral part of the design procedure for PCC pavements in the forthcoming AASHTO Guides under development (*30*).]

MATERIALS CHARACTERIZATION

An important aspect of the development of analytically-based methodologies has been the evolution of procedures to define requisite material characteristics. A number of the analysis procedures summarized in Table 1 are based on the assumption of linear response (either elastic or viscoelastic). The majority of materials used in pavement structures do not satisfy such an assumption. Accordingly, ad hoc simplifications of materials response have been used.

Materials characterization for analytically-based design methodologies requires definition of stress versus strain relationships, termed *stiffness* or *resilient modulus* for each pavement component. These moduli can be used to determine stresses, strains, and deflections within the pavement structure. As shown in Figure 12, results of such computations permit estimates of the

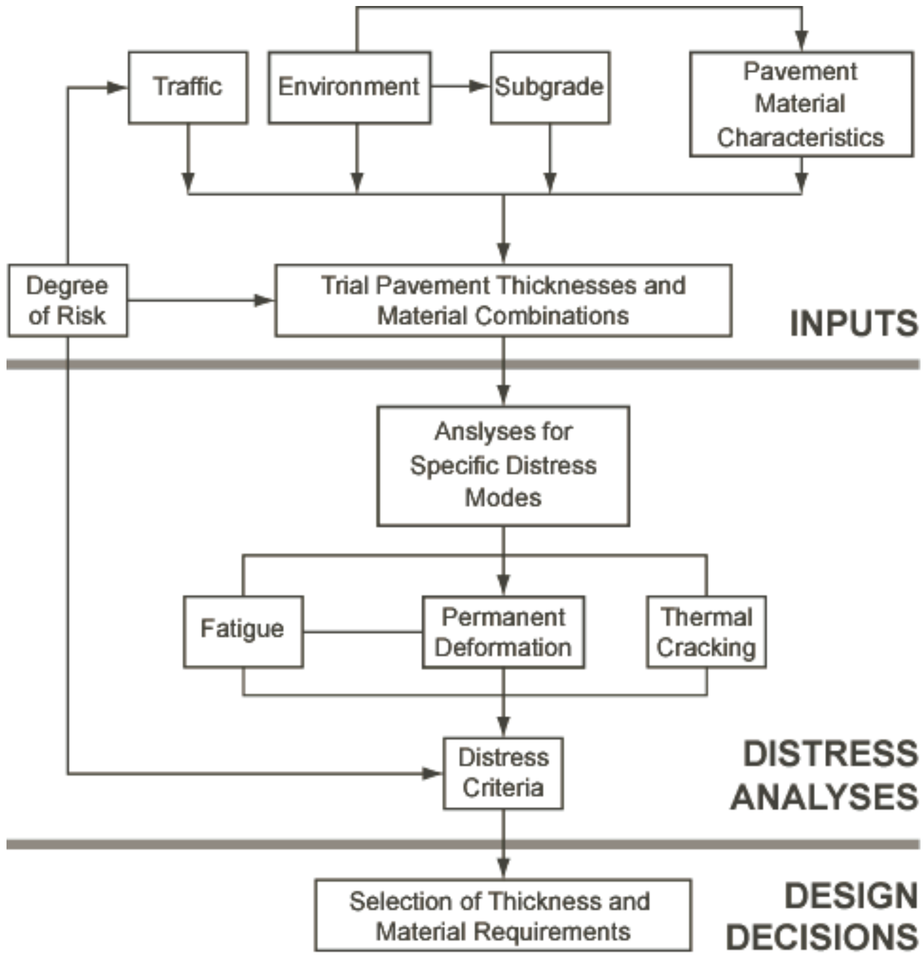

Figure 12. Simplified design/analysis framework.

various forms of distress which influence pavement performance. Table 2 summarizes methods currently used for these determinations.

Determinations of distress criteria are also part of the characterization process. For asphalt concrete pavements, these include measures of the permanent deformation, fatigue, and fracture characteristics for the treated components and the permanent deformation response of soil and untreated granular materials. As will be seen subsequently, these characteristics may be determined through laboratory testing for a specific project. Alternatively, estimated values of

Table 2 Methods Which Have Been Used to Determine Stiffness for Soil and Pavement Materials

Mode of Loading	Load Condition	Form of Load Application	Stiffness Measure	Confinement	Material		
					Asphalt Concrete	Fine-grained Soil	Untreated Granular Material
Axial (Normal Stress)	Compression, Tension (bound materials)	Creep	Compliance, creep modulus	With	X	X	X
				Without	X	--	--
		Dynamic	Complex modulus	With	X	X	X
				Without	X	--	--
		Repeated Load	Resilient modulus	With	X	X	X
				Without	X	X	--
Shear (Shear Stress)	--	Creep	Compliance, creep modulus	With	--	--	--
				Without	X	--	--
		Dynamic	Compliance	With	--	X	X
				Without	X	--	--
		Repeated Load	Resilient modulus	With	--	X	X
				Without	X	--	--
Diametral (Indirect Tensile Stress)	--	Creep	Compliance, creep modulus	Usually without	X	--	--
		Repeated Load	Resilient modulus	Usually without	X	--	--
Flexure (Flexural Stress)	--	Dynamic	Complex modulus	--	X	--	--
		Repeated Load	Flexural stiffness	--	X	--	--

response representative of specific categories of materials may be selected based on previous research data.

When defining response characteristics, service conditions must be properly considered. They include: (1) stress state—associated with loading; (2) environmental conditions—moisture and temperature; and (3) construction conditions—e.g., water content and dry density for untreated materials and degree of compaction for asphalt-bound materials. To ensure that materials evaluation is accomplished at reasonable cost, these service conditions must be carefully selected. In this section both stress-strain relationships, referred to as stiffness, and distress characteristics will be briefly summarized.

Stiffness

Developments relative to pavement materials stiffness characteristics, which include asphalt-aggregate mixes, untreated fine-grained soils and granular materials, are summarized in this section. The reader is referred to Reference (*31*) for a summary of these characteristics for portland cement-treated and lime-treated materials.

Asphalt Mixes

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

$$S_{mix} = \frac{\sigma}{\sigma}(t,T) \qquad \textbf{(1)}$$

where:

S_{mix}	=	mixture stiffness
σ , ε	=	stress and strain, respectively
t	=	time of loading
T	=	temperature

This approach was first presented by Van der Poel (*32*) in 1950, and expanded by Heukelom and Klomp (e.g., *33*). At temperatures above 25ºC, it is likely that the stress state has an influence on the stiffness characteristics of these materials, becoming more pronounced as the binder is less stiff. This effect may be reflected in an ad hoc manner when considering specific modes of distress.

Inherent in this approach is the assumption of the interchangeability of time and temperature. Such interchangeability is incorporated in a number of the design procedures to be discussed subsequently. It is useful to the design engineer since properties can be measured at one temperature for times of loading which are tractable and extended to other temperatures and times of loading which may be difficult to reproduce in the laboratory.

Mix stiffness can also be estimated from parameters such as the properties of the binders as they exist in the mix in the field and the volumetric proportions of the components. Two such examples are: 1) estimation procedures developed by the Shell investigators (*34, 35*) and incorporated in the Shell design procedure (*36*); and 2) procedure developed by Witczak (*37,* 38) and used both in the Asphalt Institute design procedure (*39*) and in the new AASHTO Guide methodology, under development (*30*).

The Shell method requires a measure of the stiffness of the binder and the volumetric properties of the aggregate, binder, and air in the compacted mix. Witczak's procedure incorporates aggregate grading characteristics as well (e.g., *38*).

Fine-Grained Soils

While there has been considerable research on the stiffness characteristics of fine-grained soils and granular materials by a number of investigators, impetus for these efforts was provided by the research of H.B. Seed (Figure 13) and his associates (*40*). The term *resilient modulus* was

Figure 13. H.B. Seed.

introduced to describe the relationships between applied stress and recoverable strain measured in a repeated load triaxial compression test. For fine-grained soils, the stiffness characteristics are dependent on dry density, water content or suction, soil structure, and stress state. For a particular condition, the stiffness as defined by resilient modulus, for example, is dependent on the applied stress; that is:

$$M_r = F(\sigma_d) \qquad \textbf{(2)}$$

where:

M_r = resilient modulus, σ_d / ε_r

σ_d = repeated applied deviator stress

ε_r = recoverable strain measured after some prescribed number of applications of σ_d

The mechanical properties of soils and granular materials depend on the effective stress state (total stress - pore water pressure); accordingly Brown et al. (*41*) (Figure 14) suggested a more general model, based on tests on saturated clays, of the form:

$$G_r = \frac{\sigma_d}{C}\left(\frac{\sigma_m}{\sigma_d}\right)^m \qquad \textbf{(3)}$$

Figure 14. S.F. Brown.

where:

G_r	=	resilient shear modulus
$'\ '_{m}$	=	mean normal effective stress
C, m	=	constants for particular soil

For partially saturated soils with degrees of saturation greater than 85 percent, soil suction can be used in place of the mean normal effective stress (*41*). To avoid unrealistically high values of G_r at low stresses, it was recommended (*41*) that the values of G_r be related to the magnitude of strain, as had been suggested by Seed and Idriss (*42*) for soil response to earthquake loads and used extensively in a variety of small strain geotechnical problems.

Figure 15 illustrates the influence of deviator stress on the response of the subgrade soil for the AASHO Road Test (*40*) while Figure 16 illustrates the dependence of the subgrade stiffness on soil moisture suction (*43*), as suggested by Equation (3).

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 17 (*44*), even though its water content may remain constant. This was originally

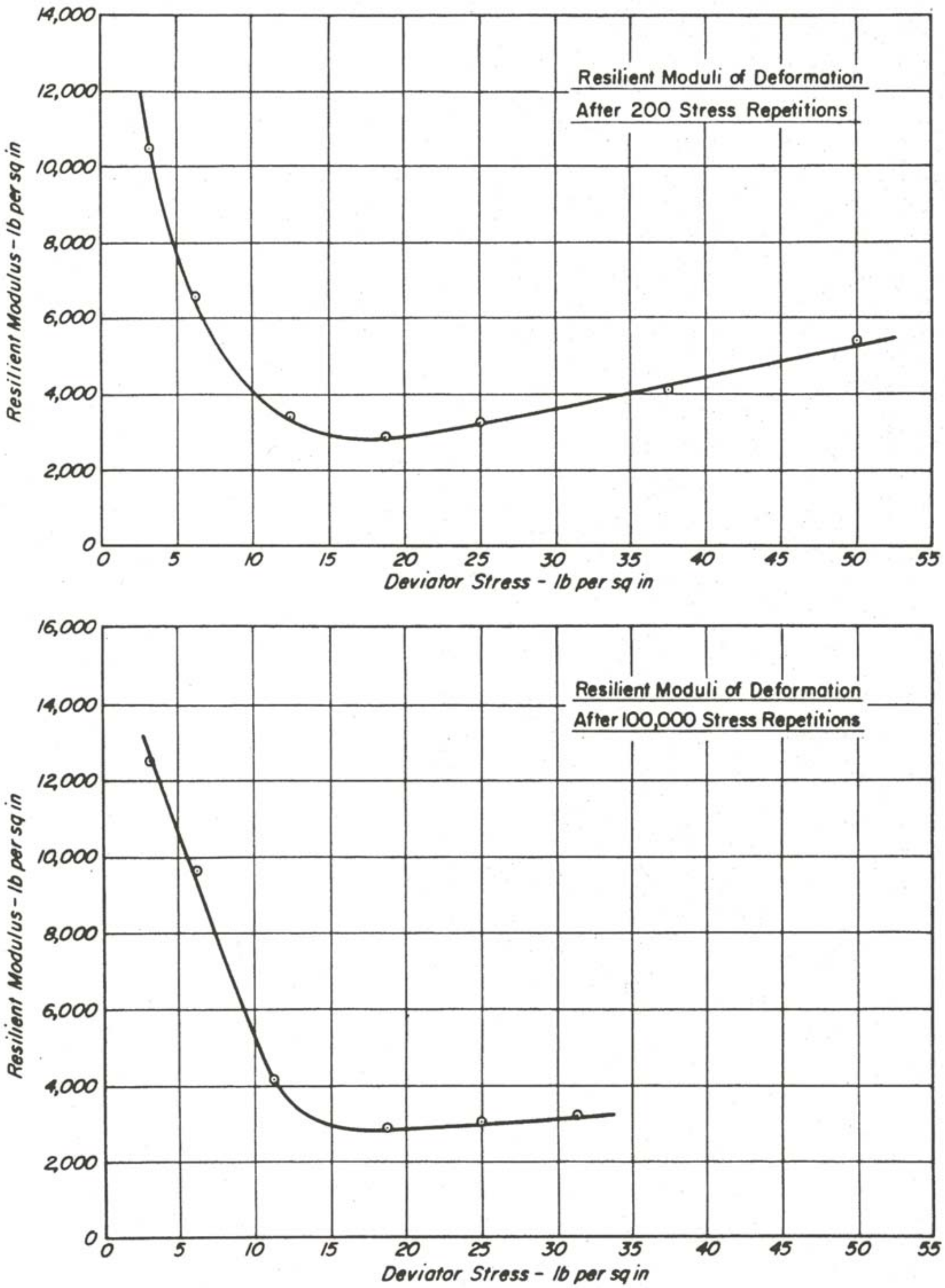

Figure 15. Results of repeated load tests, AASHO Road Test subgrade.

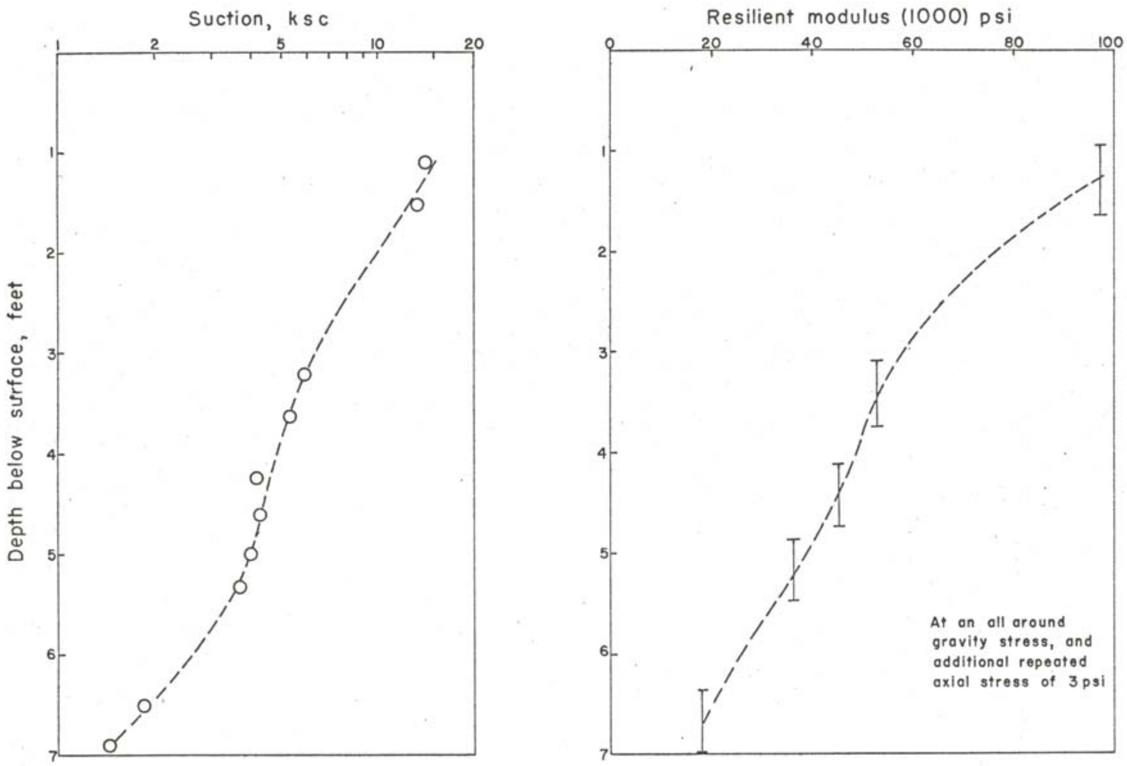

Figure 16. Relationships between suction and resilient modulus.

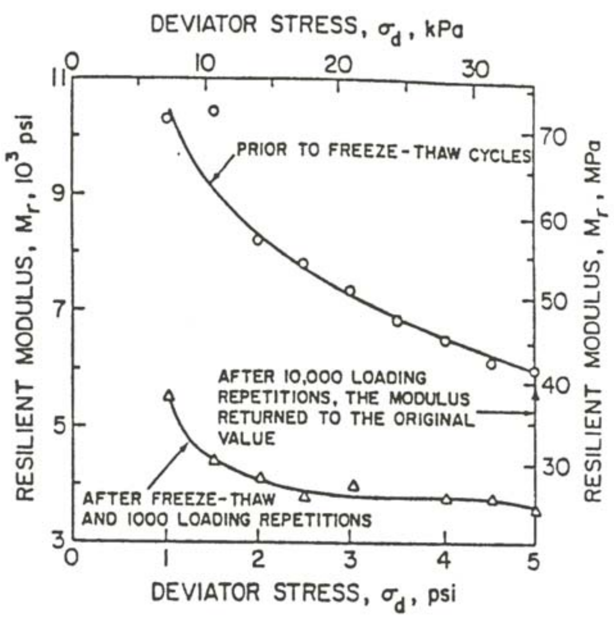

Figure 17. Resilient modulus test results before and after freeze-thaw for undisturbed regina clay.

suggested by Sauer (*45*). 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 (*46*), particularly the relationship among water content, dry density, soil structure, and method of compaction. At water contents dry of optimum for a particular compactive effort, clay particles are arranged in a random array termed a "flocculated" structure. At water contents wet of optimum (provided shearing deformations are induced during compaction), particles are oriented in a parallel fashion, often termed "dispersed" (*46*). As suggested by Lambe (*47*) and demonstrated by Mitchell (Figure 18) (*48*), 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, Figure 19 (H.B. Seed, unpublished data). The resilient response is also shown in this figure. If the designer

Figure 18. J.K. Mitchell.

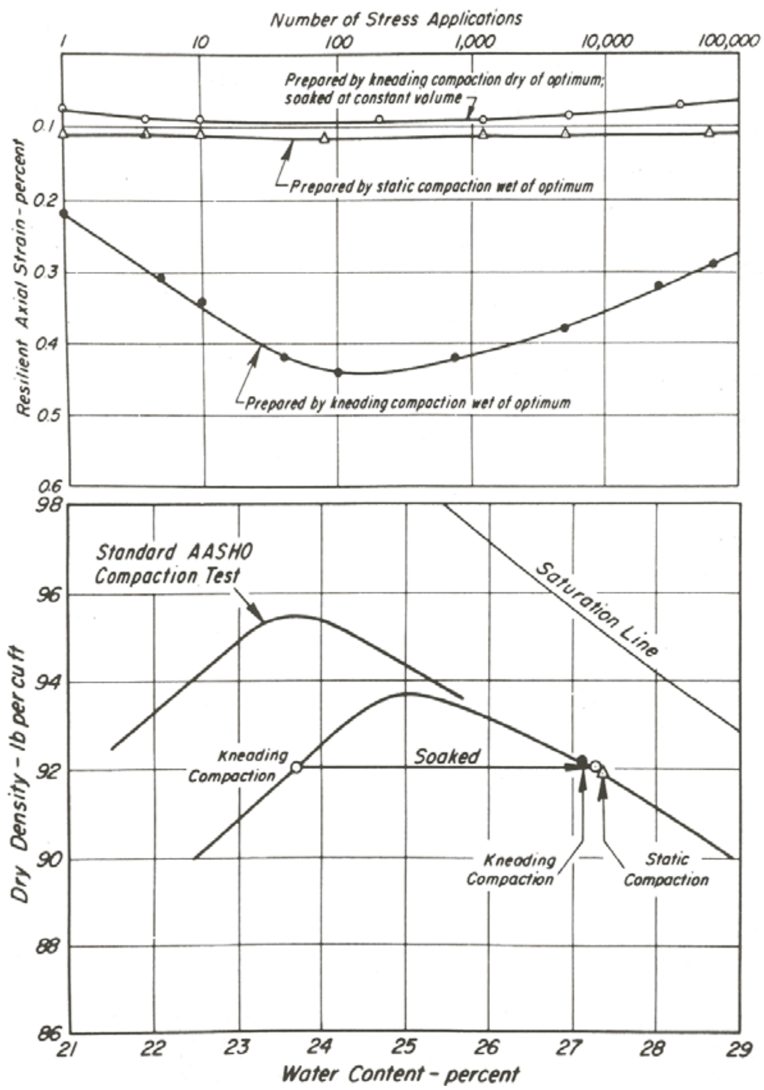

Figure 19. Influence of soil structure on resilient response (H.B. Seed, unpublished data).

compacted 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 obtained. 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 utilizes them in the selection of conditions for specimen preparation for testing. Guidelines based on such considerations are available (*49*).

If equipment is not available to measure the stiffness modulus, a number of procedures to estimate this property from other tests have been developed. One of the well known approximate relations is that developed by the Shell Investigators (*50*) which was based on correlations between dynamic in-situ tests and the corresponding measured CBR values to estimate subgrade stiffness modulus:

$$E_{sub} = 10 \cdot CBR \tag{4}$$

where:
E_{sub} = subgrade modulus, MPa
CBR = California Bearing Ratio

Note: Use of this relationship should be restricted to CBRs less than 20. In addition, it should be noted that stiffness values might range from 5 to 20 times the CBR.

Other relationships include, for example, those developed by the TRRL, also based on CBR (*51*); and by Dawson and Gomes Correia (*52*) which covers the practical range for subgrades in the UK (*52*).

Equation (4) has been used as a basis for estimation of soil modulus in a number of design procedures.

Untreated Granular Materials

The stiffness characteristics of untreated granular materials are dependent on the applied stresses. This stress dependency has been expressed in several ways for pavement design and analysis purposes (*53–56*):

$$E_{gran} = K_1 \sigma_3^{\,n} \tag{5}$$

$$E_{gran} = K_1 \theta^{k_2} \tag{6}$$

$$E_{gran} = F\left(\sigma'_m, \sigma_d\right) \tag{7}$$

where:

E_{gran} = stiffness modulus

σ_d, σ_3 = deviator stress and confining pressure in a triaxial compression test respectively;

θ = sum of principal stresses; in triaxial compression ($\sigma_d + 3\sigma_3$);

σ'_m = mean normal effective stress ($\sigma_d + 3\sigma_3$)/3; and

K, n, K_1, K_2 = experimentally determined coefficients.

Equation (5) is used in an ad hoc manner in layered elastic analyses (*39*) and in finite-element idealizations (*25*) both of which are used for pavement design and analysis purposes. Table 3 contains a summary of aggregate responses representative of the behavior depicted by Equation (5). The equation stresses the importance of effective stress and deviator stress on resilient response. Moreover, data suggest that the ratio of those stresses is most influential (*57*).

Table 3 Summary of Representative Repeated Load Triaxial Compression Test Data for Untreated Granular Materials

Material	**K_1**	**K_2**	**Reference**
Partially crushed gravel; crushed rock	1,600-5000	0.57-0.73	(*55*)
Crushed stone	4,000-9,000	0.46-0.64	(*58*)
Well-graded crushed limestone	8,000	0.67	(*56*)

*Coefficients K_1 and K_2 shown in this table when used with Equation (6)—result in modulus values in psi units.

Researchers who have contributed to the development of these stiffness relationships include, but are not limited to the following: G. Dehlen, R.G. Hicks, and R.D. Barksdale (Figure 20).

Although the method of compaction is important for fine-grained soils because of soil structure considerations, one of the primary factors affecting the stiffness characteristics of granular materials is water content (degree of saturation) since this is directly related to effective stress. As compared to fine-grained soils, method of compaction has a comparatively lesser effect on soil structure (*59*). Accordingly, any method of compaction (e.g., vibratory) that

G. Dehlen.

R.G. Hicks

R.D. Barksdale

Figure 20. G. Dehlen, R.G. Hicks, and R.D. Barksdale.

produces the desired dry density at the appropriate water content may be considered suitable for laboratory preparation of specimens for testing.

An alternative approach is to test the aggregate in a dry state (when pore pressures are zero) and select design values from estimates of the suction based on anticipated worst case drainage conditions.

As with fine-grained soils, estimates of the stiffness characteristics of granular materials may be utilized. The Shell investigators suggest that since granular materials will only sustain very small tensile stresses, the ratio of the modulus of the granular layer to that of the subgrade is limited to a range of about 2 to 3 (*36*). Work by Brown and Pappin (*60*) using finite element analysis and a more detailed non-linear model for granular material response, suggested for British conditions a design value of 100 MPa for good quality granular materials in an unsaturated state with lower values for poorer quality materials in saturated conditions. Experience over a number of years with back-analysis of Falling Weight Deflectometer data from a wide range of pavements has confirmed that this value is appropriate for the conditions they encountered.

Strength (Bearing Capacity).

Prior to 1962, considerable effort was devoted to consideration of bearing capacity, e.g. the work of Nijboer (Figure 21) and Saal (*61, 62*), McLeod (Figure 22) (*63*), and Smith (*64*). Using triaxial tests on asphalt mixes, values of cohesion, *c,* and angle of internal friction, ϕ, for specific temperatures and rates of loading were defined using bearing capacity relationships such as that developed by Prandtl for a continuous strip loading (*61*):

$$q_{ult} = cF(\phi) \quad \textbf{(8)}$$

Figure 21. L.W. Nijboer.

Figure 22. N.W. McLeod.

where:

q_{ult} = Bearing capacity, psi or kg per cm^2,

$F(\phi)$ = Function dependent on ϕ; e.g., for ϕ = 25º, $F(\phi)$ = 20.7.

When q_{ult} is made equal to a specific contract pressure, c and ϕ are related as shown in Figure 23. In this figure, a mix with a value of c and ϕ lying on or to the right of the Nijboer curve would be adequate for vehicles equipped with 100-psi tires.

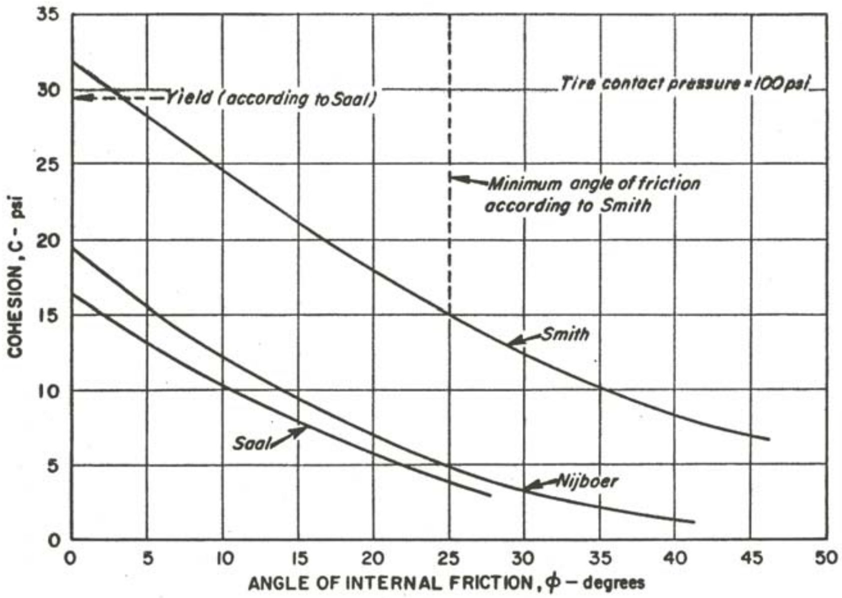

Figure 23. Relationship between cohesion and angle of internal friction to prevent plastic flow or overstress at a particular point in asphalt mixture.

Saal (*62*) has suggested modification of this relationship recognizing that the bearing capacity for a circular area is larger than that for a continuous strip. The results of Nijboer and Saal shown in Figure 23 were considered applicable for standing loads.

Smith (*64*) has presented a relationship between c and ϕ and bearing capacity for a circular area based on a yield criterion rather than a plastic flow condition as in the above formulations. For the same contact pressure, larger values of c and ϕ are required than in the previous case, as seen in Figure 23. Smith also suggested a minimum angle of friction of 25º to minimize the development of instability from repeated loading. His relationship was considered suitable for moving traffic.

While these relationships have not been used recently, the work of Nijboer and Saal would still appear to be useful for static loading conditions.

In addition to examining bearing capacity under vertical loading (*63*), McLeod also developed useful information pertaining to decelerating or accelerating loads. Results of one such analysis by McLeod (*63*) for a load with a contact (or tire) pressure of 100 psi are presented in Figure 24, including the equation on which the curves are based. The terms *P* and *Q* are measures of friction between a tire and pavement and pavement and base respectively. The curves A and B in this figure indicate the importance of pavement thickness in minimizing this form of instability when a frictionless contact between asphalt concrete surfacing and base is assumed (P – Q = 1). As the asphalt concrete thickness increases, the ratio ℓ/t (ratio of length of

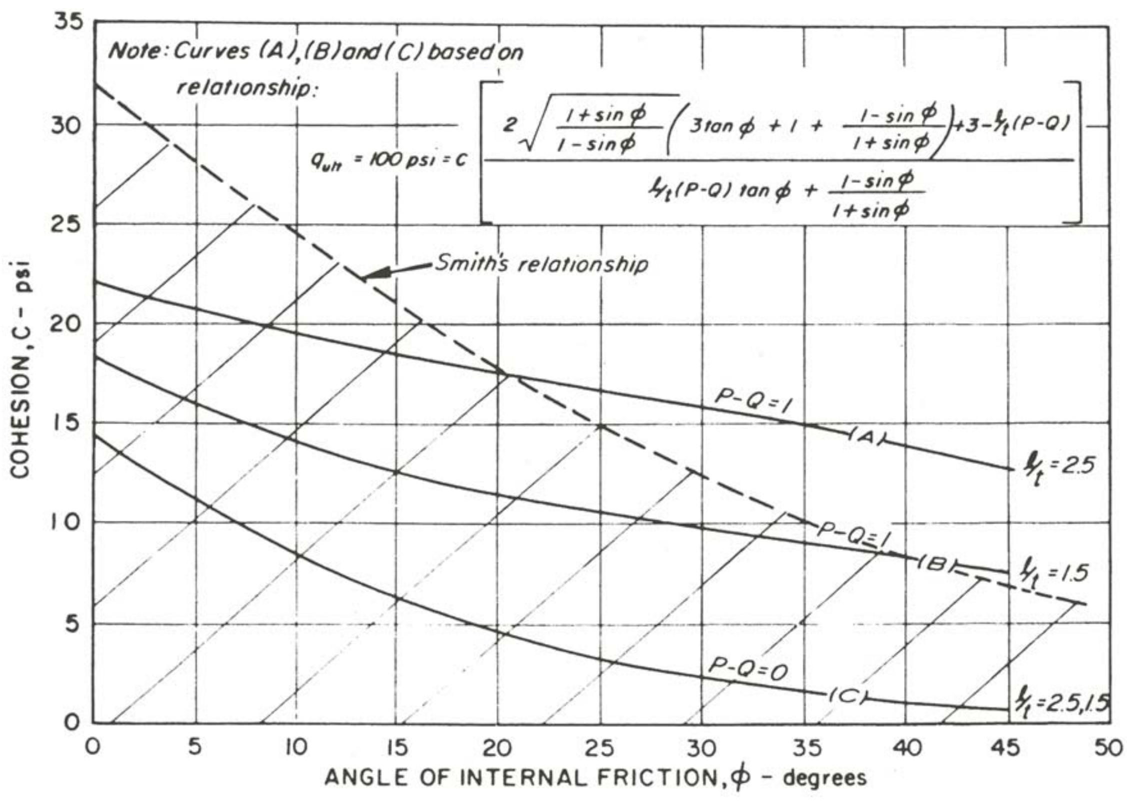

Figure 24. Stability curves for asphalt mixtures subject to braking stresses. (After McLeod)

tire tread to asphalt concrete thickness) decreases, resulting in lower values of c at a given ϕ to prevent instability.

When P – Q = 0 (full friction between pavement and base—a more practical situation in well-designed and constructed pavements) and the thickness of the asphalt concrete is in the range of 4 to 6 in. (Curve C), the more critical conditions are defined by the curve suggested by Smith, as shown in Figure 24.

Permanent Deformation Attributed to Unbound Materials

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. For well compacted asphalt concrete, available information suggests that shear deformation rather than densification is the primary rutting mechanism (*65*).

From a pavement design standpoint, a number of approaches have evolved to consider rutting. The first considers limiting the vertical compressive strain at the subgrade surface to a value associated with a specific number of load repetitions, this strain being computed by means of layered elastic analysis. The logic of this approach, first suggested by the Shell researchers, e.g. G. Dormon (Figure 25) (*16*), is based on the observation that, for materials used in the pavement, permanent strains are proportional to elastic strains.[5] By controlling the elastic strain to some prescribed value, the plastic strain will also be limited. Integration of the permanent strains over the depth of the pavement section provides an indication of the rut depth. By

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[5] Laboratory test data on soils, aggregates, and asphalt mixes support this assumption [e.g., Reference (*66*)].

Figure 25. G.M. Dormon.

controlling the magnitude of the elastic strain at the subgrade surface, the magnitude of the rut is thereby controlled.

An equation of the following form is used to relate the number of load applications to vertical compressive strain at the subgrade surface:

$$N = A\left(\frac{1}{\varepsilon_v}\right)^b \qquad \textbf{(9)}$$

where:

N	=	number of load applications,
ε_v	=	elastic vertical strain at subgrade surface,
A, b	=	empirically determined coefficients

This approach has been quantified by the back-analysis of pavements with known performance, but is semi-empirical in nature since it applies to a particular range of structures with particular materials under particular environmental conditions. Values of the coefficients have been derived for different locations and circumstances. For example, the value for the exponent *b* is in the range 0.22 to 0.27.

Figure 26 illustrates Equation (9) schematically. In this figure it will be noted that beyond about 50×10^6 ESALs, Equation (9) may exhibit a flatter slope (*67*). For long life pavements, it is likely that this change in slope should be recognized to avoid overly conservative thick structural pavement sections for repetitions in excess of 50×10^6 ESALs.

A number of investigations have suggested that an alternative approach to control rutting in the unbound layer is to limit the vertical compressive stress at the surface of that layer, e.g., Thompson (Figure 27) (*68*) and Maree (*69*).

Fatigue Cracking in Asphalt Concrete.

Considerable research has been devoted to fatigue cracking in asphalt concrete. Results of this research have demonstrated that the fatigue response of asphalt concrete to repetitive loading can be defined by relationships of the following form (*70–74*):

$$N = A\left(\frac{1}{\varepsilon_t}\right)^b \quad \textbf{or} \quad N = C\left(\frac{1}{\sigma_t}\right)^d \qquad \textbf{(10)}$$

where:

N = number of repetitions to failure,

ε_t = magnitude of the tensile strain repeatedly applied,

σ_t = magnitude of the tensile stress repeatedly applied, and

A, b, C, d = experimentally determined coefficients.

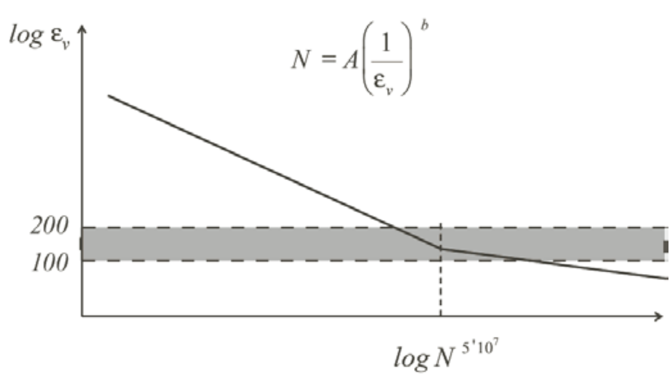


Figure 26. Schematic of surface vertical compressive strain versus load repetitions.

Organizations contributing early to this area included the University of Nottingham, e.g., the work of Pell, Figure 28, (*75*) and the University of California, Berkeley (*76*). Others have included: Shell, both in the Netherlands (KSLA) (*77*) and France (*78, 79*); The Laboratoire Central Ponts at Chaussees (LCPC) (*80*); the TRL in the UK (*81*); CSIR, South Africa (*82*); RR, Belgium (*83*) and the Asphalt Institute, U.S (*84*). Examples of fatigue test equipment currently in use are illustrated in Figure 29.

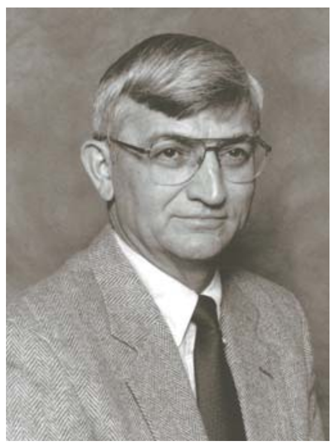
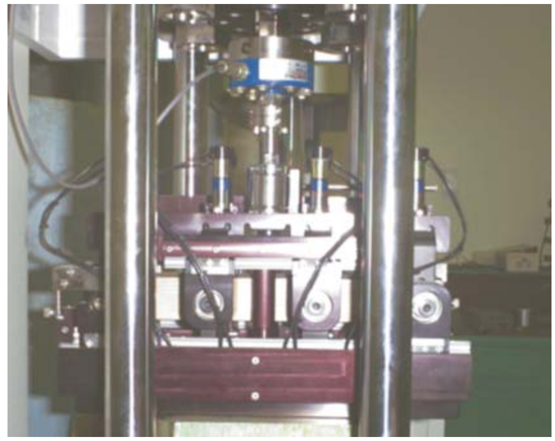


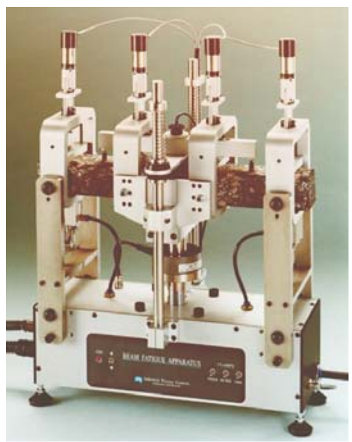
Figure 27. M. Thompson.

Figure 28. P.S. Pell.

a. LCPC and University of Nottingham equipment.



b. SHRP-developed equipment.



c. Australian beam-fatigue apparatus (based on the SHRP equipment design).

Figure 29. Fatigue test equipment currently in use.

A number of factors influence fatigue response as measured in the laboratory including the mode of loading (controlled stress or load and controlled strain or deformation (*85, 70*).

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

$$N = K\left(\frac{1}{\varepsilon_t}\right)^a \left(\frac{1}{S_{mix}}\right)^b \tag{11}$$

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

$$\frac{V_{asp}}{V_{asp} + V_{air}} \tag{12}$$

where V_{asp} is the volume of asphalt and V_{air} is the volume of air. Data developed by a number ofresearchers (e.g., *72, 73*) have permitted the quantification of Equation (12), for example, in the Asphalt Institute design procedure (*39*).

Equation (11) is used in the Shell (*36*) and Asphalt Institute (*39*) procedures with the coefficients set according to the amount of cracking considered tolerable, the type of mixture that might be used, and the thickness of the asphalt-bound layer. A few examples of fatigue design relationships following the form of Equation (11) are shown in Table 4.

An alternative approach to define fatigue response makes use of the concept of dissipated energy suggested by Chompton and Valayer (*86*) and van Dijk (*77*). The form of the relationship between the load repetitions and dissipated energy is:

$$WD = AN^Z \tag{13}$$

where:

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

A number of researchers have utilized Equation (13) in lieu of Equation (11) as the damage determinant. To do this requires the use of viscoelastic rather than elastic analysis, and as of this date has not been widely practiced (*87*).

Table 4 Design Fatigue Relationships

Design Method	Design Relationship	Equation	Remarks
NCHRP1-10B [Finn et al. (*20*)]	Greater than 45 percent wheel path cracking: $\log_{10} N(ε\ 45\%) = 16.086\ ε\ 3.291 \log_{10} ε \frac{\varepsilon_t}{10^{ε 6}} ε\ 0.854 \log_{10} ε \frac{S_{mix}}{10^3} ε$	**(15)**	ε_t = tensile strain in./in. × 10^{-6}
	For less than 10 percent wheel path cracking: $\log_{10} N(\leq 10\%) = \leq \frac{\log_{10} N(\leq 45\%)}{1.4} \leq$	**(16)**	S_{mix} = mix stiffness, psi
The Asphalt Institute (*39*)	$[\log_{10} N(\geq 45\%)] \geq C$	**(17)**	Makes use of NCHRP 1-10B equation for 45 percent in more wheel path cracking modified by factor C to reflect the effect of asphalt content and air-void content
	$C = 10^M$	**(18)**	
	$M = 4.84 \left(\frac{V_{asp}}{V_{asp} + V_{air}} - 0.69 \right)$	**(19)**	
Shell International (*36*)	$N = SF(K_{1r}) α \left(\frac{1}{\alpha_t} \right) α \left(\frac{1}{S_{mix}} \right) α$	**(20)**	
	$(K_{1r}) = f$ (mix volumetrics, pen. index of asphalt)		
	$\alpha = f$ (AC thickness)		
	$K_3 = f$ (AC thickness)		

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 is the linear summation of cycle ratios (sometimes referred to as Miner's hypothesis) (*71*). This was originally suggested by Peattie in 1960 based on his wok on the fatigue of metals. The linear summation of cycle ratios hypothesis is stated as follows:

$$\overset{n}{\underset{i=1}{\leq}} \frac{n_i}{N_i} \leq 1 \qquad \textbf{(14)}$$

where:

n_i = the number of actual traffic load applications at strain level *i,* and

N_i = 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.

Permanent Deformation (Rutting) in Asphalt Concrete Layer(s).

Two methodologies are briefly described in this section. The first approach, originally suggested by Heukelom and Klomp (*33*) and Barksdale (*88*) and used by MacLean (*61*) and Freeme (*89*) (Figure 30), makes use of elastic analysis to compute stresses within the asphalt-bound layer together with 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, Figure 31.

One version of this approach was developed by the Shell researchers and has been used in modified form to evaluate specific pavement sections (*36*). In this methodology, creep test results are incorporated into the following expression to estimate rutting:

$$\Delta h_1 = C_m \tag{21}$$

$$\sigma h_1 = C_m \sum_{i=1}^{n} h_{1\sigma i} \frac{(\sigma_{ave})_{1\sigma i}}{(S_{mix})_{1\sigma i}} \tag{22}$$

Figure 30. C.R. Freeme.

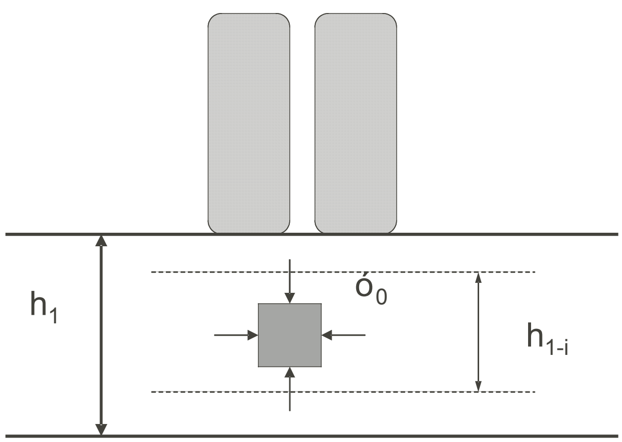


Figure 31. Permanent deformation prediction—layered strain procedure.

where:

Δh_1 = permanent deformation in the asphalt-bound layer,

$h_{1=i}$ = thickness of sublayer of asphalt-bound layer with thickness

$(\sigma_{ave})_{1\sigma i}$ = average vertical stress in layer, and

$(S_{mix})_{1=i}$ = mix stiffness for layer 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. If it is planned to use mixes containing modified binders, use of creep test data for these mixes in Equation (22) will not provide correct estimates of mix performance since mixtures containing these binders behave differently under loading representative of traffic as compared with their behavior in creep. For example, the work of the Shell investigators suggest that use of creep test data may over predict rutting for mixes containing some modified binders (*90*).

To consider the effects of stresses of different magnitudes on the development of rutting, which result from variations in traffic loads and environmental condition, a cumulative damage hypothesis is required, just as for fatigue. A “time-hardening” procedure suggested by Freeme (*89*) appears to provide a reasonable approach. The methodology is illustrated schematically in Figure 32 for two applied stresses with different magnitudes.

For the first load block (smaller load), the specimen deforms as shown in Figure 32. The equivalent number of repetitions of the larger load is shown in the figure at the same strain level. For the second load block, the deformation follows the path along the curve for the larger load from the strain caused by the first load block. This procedure is repeated, moving from curve to curve for successive applications of the two load magnitudes. The procedure used in the proposed AASHTO guide makes use of this approach (*30*).

The second approach was developed by Deacon and used to analyze the results of rutting in 34 of the 36 WesTrack test sections (*91*).

The pavement is represented as a multilayer elastic system and the analysis consists of determining three parameters, $\propto$, $\propto^{e}$, and $\propto_{v}$[6] on an hour-by-hour basis. Measured temperature distributions are used to define the moduli of the asphalt concrete which was subdivided into a number of layers from top to bottom with thicknesses 25 mm (1 in.), 50 mm (2 in.) for the first and second layers, and convenient thicknesses for the remainder of the asphalt concrete layer, Figure 33, to simulate the effects of temperature gradients on mix stiffness.

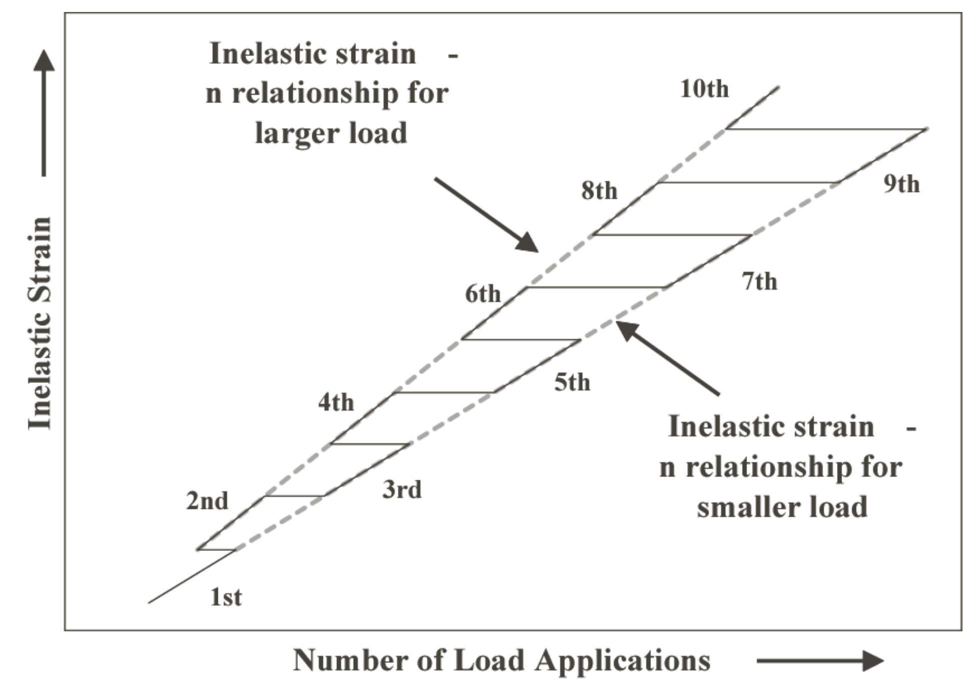


Figure 32. Compound loading—time hardening.

[6] $\propto$, $\propto^{e}$ = elastic shear stress and strain at a depth of 50 mm (2 in.) below outside edge of tire
$\propto_{v}$ = elastic vertical compressive strain at the subgrade surface

In this model, rutting in the asphalt concrete is assumed to be controlled by shear deformations. Accordingly, the computed values for $\propto$and $\propto^{e}$ at a depth of 50 mm (2 in.) beneath the edge of the tire were used for the rutting estimates. Densification of the asphalt concrete is excluded in these estimates since it has a comparatively small influence on surface rutting if the asphalt concrete layer is compacted to an air-void content not exceeding 8 percent.

In simple loading, permanent shear strain in the AC is assumed to accumulate according to the following expression:

$$\cdot^{i} = a \cdot \exp\left(b \cdot \right) \cdot^{e} n^{c} \qquad \textbf{(23)}$$

where:

γ^{i}	=	permanent (inelastic) shear strain at 50 mm (2 in.) depth
Δ	=	shear stress determined at this depth using elastic analysis
γ^{e}	=	corresponding elastic shear strain
n	=	number of axle load repetitions
a, b, c	=	regression coefficients

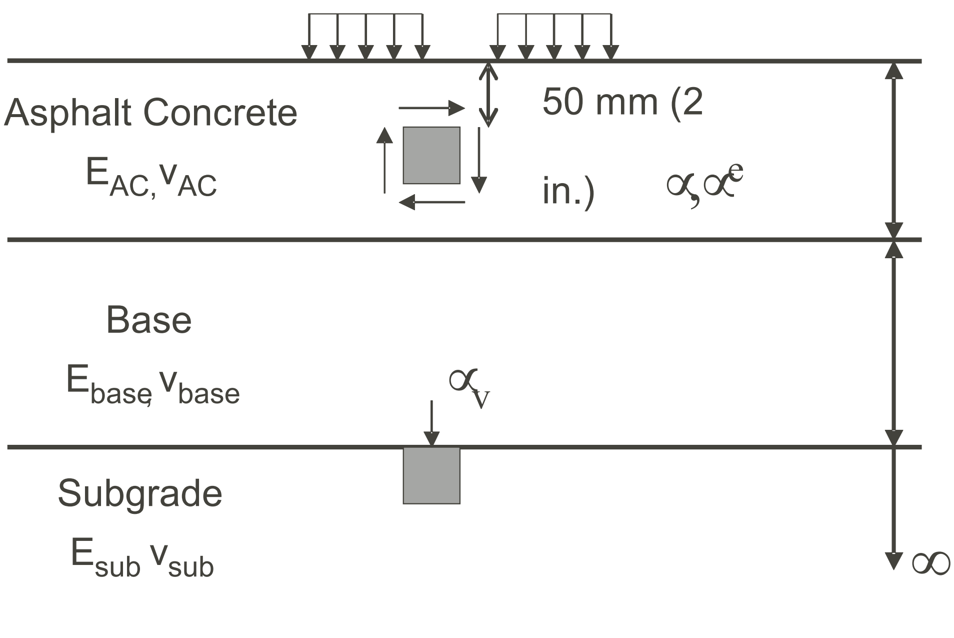


Figure 33. Permanent deformation prediction—use of shear stress and strain.

The time-hardening principle is used to estimate the accumulation of inelastic strains in the asphalt concrete for in-situ conditions. The resulting equations are as follows:

$$a_j = a \cdot \exp(b \cdot \ ) \cdot {}^{e}_{j} \quad \textbf{(24)}$$

$$\gamma_1^i = a_1 [\gamma \, n_1]^c \quad \textbf{(25)}$$

$$\gamma_j^i = a_j \left[\left(\gamma_{j\gamma 1}^i / a_j \right)^{(1/c)} + \gamma \, n_1 \right]^c \quad \textbf{(26)}$$

where:

j	=	j^{th} hour of trafficking
γ_j^e	=	elastic shear strain at the j^{th} hour
Δn	=	number of axle load repetitions applied during the j^{th} hour

The concept is illustrated schematically in Figure 32.

Rutting in the AC layer due to the shear deformation is determined from the following:

$$rd_{AC} = K\gamma_j^i \quad \textbf{(27)}$$

For a thick asphalt concrete layer, the value of K has been determined to be 10 when the rut depth (rd_{AC}) is expressed in inches (*65*).

To estimate the contribution to rutting from base and subgrade deformations, a modification to the Asphalt Institute subgrade strain criteria, i.e., a modification of Equation (9), can be utilized (*91*).

MECHANISTIC EMPIRICAL DESIGN AND ANALYSIS

Currently, there are many mechanistic-empirical (analytically-based) design procedures which have been developed. Some, while not used, have served as the basis for other procedures. Several such procedures are briefly summarized in Table 5. Figure 12 illustrates a simplified framework which the procedures generally follow. All the procedures idealize the pavement structure as a multilayer elastic or viscoelastic system using programs like those described in Table 1.

While the procedures listed in Table 5 all received impetus from the 1962 Conference (as noted earlier), the U.S. Navy was using a pavement design procedure in the 1950's for airfield pavements which incorporated results of Burmister's solution for a two-layer elastic solid. A plate bearing test was used to measure the subgrade modulus and the thickness required was based on the requirement that the computed surface deflection not exceed 5 mm (0.2 in.) for the specific aircraft.

The procedures listed in Table 5 all consider the fatigue and rutting modes of distress in establishing pavement structures. Fatigue estimates are based on relationships like those shown in Table 4 and on subgrade strain or stress criteria. Some procedures utilize a layer strain procedure to estimate surface rutting contributed by the individual layers.

The linear sum of cycle ratios cumulative damage hypothesis is used in the majority of the methods to assess the effects of mixed traffic and environmental influences on fatigue cracking. Those procedures using a subgrade strain procedure incorporate a form of the linear sum of cycle ratios (based on compressive strain) for the same purpose. A few of the methods make use of the time-hardening procedure to estimate the cumulative effects of traffic and environment on rutting in the asphalt concrete [e.g., the Shell International and the proposed AASHTO Guide (still under evaluation as of 2004) methods.]

Some of the people associated with the development of these methods are shown in Figures 34, 35, 36, 37, and 38.

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 (*36, 90, 92, 93*)	Multilayer elastic solid	fatigue in treated layers; rutting: · subgrade strain · estimate in asphalt bound layer	temperature	asphalt concrete, untreated aggregate, cement stabilized aggregate	design charts; the computer program BISAR is used for analysis
National Cooperative Highway Research Program (NCHRP) Project 1-10B Procedure (AASHTO) (*20*)	Multilayer elastic solid	fatigue in treated layers; rutting	temperature	Asphalt concrete, asphalt stabilized bases, untreated aggregates	Design charts; computer program (MTC093)
The Asphalt Institute, Lexington, KY (MS-1, MS-11, MS-23) (*39, 94, 95*)	Multilayer elastic solid	Fatigue in asphalt treated layers; Rutting: · subgrade strain	Temperature, freezing and thawing	Asphalt concrete, asphalt emulsion, treated bases, untreated aggregate	Design charts; computer program DAMA
Laboratoire Central de Ponts et Chaussées (LCPC) (*96, 97*)	Multilayer elastic solid	Fatigue in treated layers; rutting	Temperature	Asphalt concrete, asphalt-treated bases, cement stabilized aggregates, untreated aggregates	Catalogue of designs; computer program (ELIZE) for analysis
Centre de Recherches Routieres, Belgium (*98*)	Multilayer elastic solid	Fatigue in treated layers; rutting	Temperature	Asphalt concrete, asphalt-stabilized bases, untreated aggregates	Design charts; computer program (MTC093)
National Institute for Transportation and Road Research (NITRR) South Africa (*99, 100, 101*)	Multilayer elastic solid	Fatigue in treated layers; rutting: · subgrade strain · shear in granular layers	Temperature	Gap-graded asphalt mix, asphalt concrete, cement-stabilized aggregate, untreated aggregate	Catalogue of designs; computer program
National Cooperative Highway Research Program (NCHRP) Project 1-26 Procedure (AASHTO) (*102*)	Finite element idealization; multilayer elastic solid	Fatigue in treated layers; rutting: · subgrade strain	Temperature	Asphalt concrete, untreated aggregates	ILLI-PAVE; elastic layer programs (e.g., ELSYM)
Federal Highway Administration U.S. DOT, Washington, D.C. (*103*)	Multilayer elastic or viscoelastic solid	Fatigue in treated layers; Rutting: · estimate at surface Serviceability (as measured by PSI)	Temperature	Asphalt concrete, cement stabilized aggregate, untreated aggregate, sulphur-treated materials	Computer program: VESYS
University of Nottingham, Great Britain (*104, 105*)	Multilayer elastic solid	Fatigue in treated layers; rutting: · subgrade strain	Temperature	Continuous or gap-graded asphalt mixes of known volumetrics on standard UK materials	Design charts; computer program (ANPAD) for analysis and design
Austroads (*106*)	Multilayer elastic solid	Fatigue in treated layers; rutting: · subgrade strain	Temperature, moisture	Asphalt concrete, untreated aggregates, cement stabilized aggregates	Design charts, computer program CIRCLY
National Cooperative Highway Research Program (NCHRP) Project 1-37A (Proposed AASHTO Guide) (*30*)	Multilayer elastic	Fatigue in treated layers; rutting: · subgrade strain · asphalt concrete, time hardening Low temperature cracking	Temperature, Moisture	Asphalt concrete, untreated aggregates, chemical stabilized materials	Computer program JULEA

Figure 34. W. (Pim) Visser, Shell Procedure.

M. W. Witczak

L.E. Santucci

J. F. Shook

Figure 35. The Asphalt Institute Procedure.


J. Bonizer


R. Sauterey


Ph. Leger


J. Bonnot

Figure 36. The LCPC Procedure.

Figure 37. J. Verstraeten, Belgium, Centre de Recherches Routieres Procedure.

S. Kuhn

N. Walker

H. Maree

Figure 38. South Africa, CSIR procedure

NON-DESTRUCTIVE TESTING

A key element for pavement rehabilitation, particularly overlay design, is the non-destructive evaluation of the existing pavement. The Benkelman Beam, developed during the WASHO Road Test (*5*) by A.C. Benkelman (Figure 39) has played a major role in the evolution of overlay design since it provided an inexpensive and reliable device to measure the surface deflection of a pavement under a standard load representative of actual traffic.

Mechanization of the Benkelman Beam concept to accelerate field deflection measurements was accomplished by the French with the introduction of the LaCroix Deflectograph and by Hveem with the Traveling Deflectometer (*108*). As noted above, this type of equipment has played a major role in overlay design for asphalt pavements.

Notable among the methods (*109*) are those developed by the Asphalt Institute (Benkelman Beam measured deflections) (*110*), the TRRL procedure developed by N.W. Lister (Figure 40) (Benkelman Beam and TRRL Deflectograph deflections)[7], and the State of California Procedure (Benkelman Beam and Traveling Deflectometer deflections) (*108*). The

Figure 39. A.C. Benkelman.

[7] Also used for a number of years in South Africa by the CSIR (*112*) and in Australia (*113*).

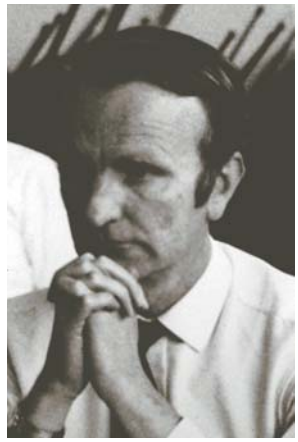


Figure 40. N.W. Lister.

work of Lister is particularly noteworthy in this regard in that he reintroduced the concept of probability of achieving a given life in the overlay with thickness requirements based on probabilities of 50 and 90 percent of achieving the design life.

In addition to the devices noted above which measure pavement response to slowly moving loads, equipment has been introduced over the years that utilizes steady-state vibratory equipment and falling weight (impulse) loading equipment (Table 6).

The vibratory equipment introduced by the Shell investigators (van der Poel and Nijboer) [e.g., Reference (*114*)] to measure dynamic pavement deflections was later extended to wave propagation measurements by Heukelomp and Klomp (*115*) (Figure 41) and Jones and Thrower of the TRL (*116–118*). The work by Jones and Thrower is particularly important in that they used different vibratory equipment over a range in frequencies to measure waves of different types (compression, shear, Rayleigh, and Love waves) and developed methodology to determine which type of wave was being measured. This, in turn, permitted estimates of both shear (G) and elastic (E) moduli of the various layers of a pavement system.

Table 6 Examples of Deflection Measuring Devices and Wave Propagation Equipment

<table>
<tr><th>Load Application</th><th>Device</th><th>Remarks</th></tr>
<tr><td rowspan="3">Slowly moving wheel load (dual tires)</td><td>Benkelman Beam (deflection beam)</td><td>Developed at WASHO Road Test</td></tr>
<tr><td>Traveling Deflectometer (California)</td><td>Developed by F.N. Hveem in California</td></tr>
<tr><td>Deflectograph</td><td>Developed in France; used e.g., in South Africa, UK, and Australia</td></tr>
<tr><td rowspan="4">Vibratory load, steady state</td><td>Dynaflect</td><td>Developed in Texas</td></tr>
<tr><td>Heavy Vibrator [U.S. Army Corps of Engineers, Waterways Experiment Station (WES)]</td><td>Developed at WES for airfield pavement evaluation (119)</td></tr>
<tr><td>Heavy Vibrator</td><td>Developed by Shell</td></tr>
<tr><td>Vibratory equipment for wave propagation; mass dependent on frequency</td><td>TRL, Shell</td></tr>
<tr><td>Falling weight (impulse load)</td><td>Falling weight deflectometer (FWD)</td><td>Developed in France and Denmark</td></tr>
</table>

Figure 41. A.J.G. Klomp

Today, the falling weight deflectometer (FWD) equipment is used extensively for surface deflection measurements as a part of overlay pavement design. Various analytical procedures to interpret the resulting measurements, referred to as back-calculation procedures, are summarized in Table 7. The FWD equipment, originally developed by the French (as seen in the timeline in Figure 1), received impetus for general use by Danish and Shell investigators [e.g., Reference (*120*)]. Table 8 lists a number of analytically-based (mechanistic-empirical) overlay design procedures developed over the years. It will be noted that the majority make use of the FWD.

Table 7 Examples of Back-Calculation Procedures

Procedure	Organization	Computer Programs	Back-Calculation Method
EVERCALC (*121*)	WashDOT	Chevron	Iterative
BISDEF (*122*)	USACE-WES	BISAR	Iterative
ELMOD (*123*)	Dynatest	MET	
MODCOMP (*124*)	FHWA	Chevron	Iterative
MODULUS (*125*)	Texas Transportation Institute	WESLEA	Iterative
PADAL (LEAD, FEAD) (*126, 127*)	University of Nottingham	PADAL, LEAD (BISAR), FEAD (FENLAP)	Iterative

IMPROVED CONSTRUCTION PRACTICES

As traffic and loadings continue to increase on our street and highway, airfield, and port and cargo transfer pavements, it is imperative that construction practices be improved to keep up with the demand. Results of mechanistic-empirical analyses allow engineers to establish such requirements. Figures 42 and 43 illustrate the impacts improved compaction and thickness control have on pavement performance expressed in terms of fatigue cracking. The results in Figure 42 illustrate that by improving compaction (lower air-void contents) and reducing variability in air-void content (as measured by the standard deviation), improved performance

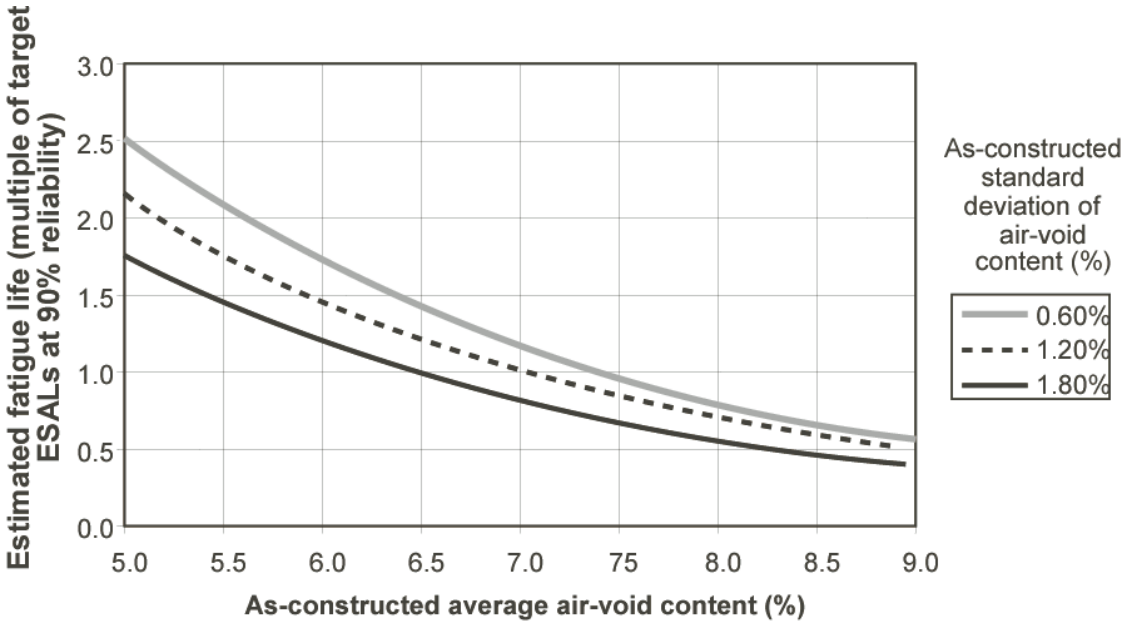


Figure 42. Effects of as-constructed air-void content on pavement fatigue performance.

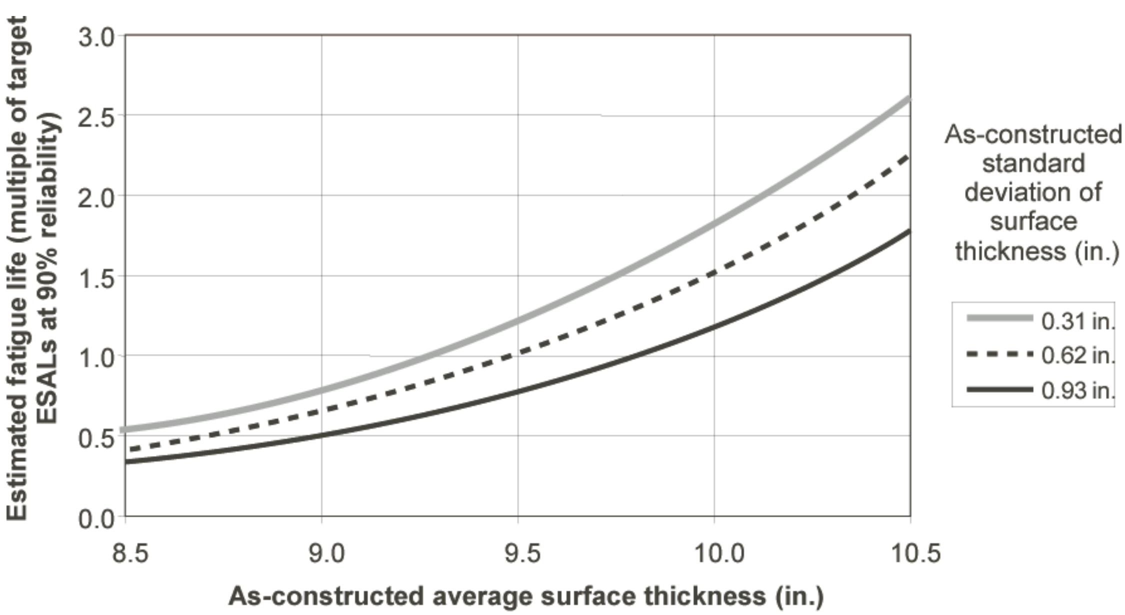


Figure 43. Effect of as-constructed surface thickness on pavement fatigue performance.

results. Similarly, the results of Figure 43 demonstrate not only the influence of thickness on fatigue performance, but also the influence of variability as well. For a given target thickness, the lower the standard deviation in constructed thickness, the better the mix performance. These as well as other construction impacts for asphalt concrete construction are described in Reference (*129, 130*).

Analyses of this type also permit one to evaluate the influence of the relative effects of variance in mix and thickness due to construction and the variance associated with testing (Figure 44) (*131*).

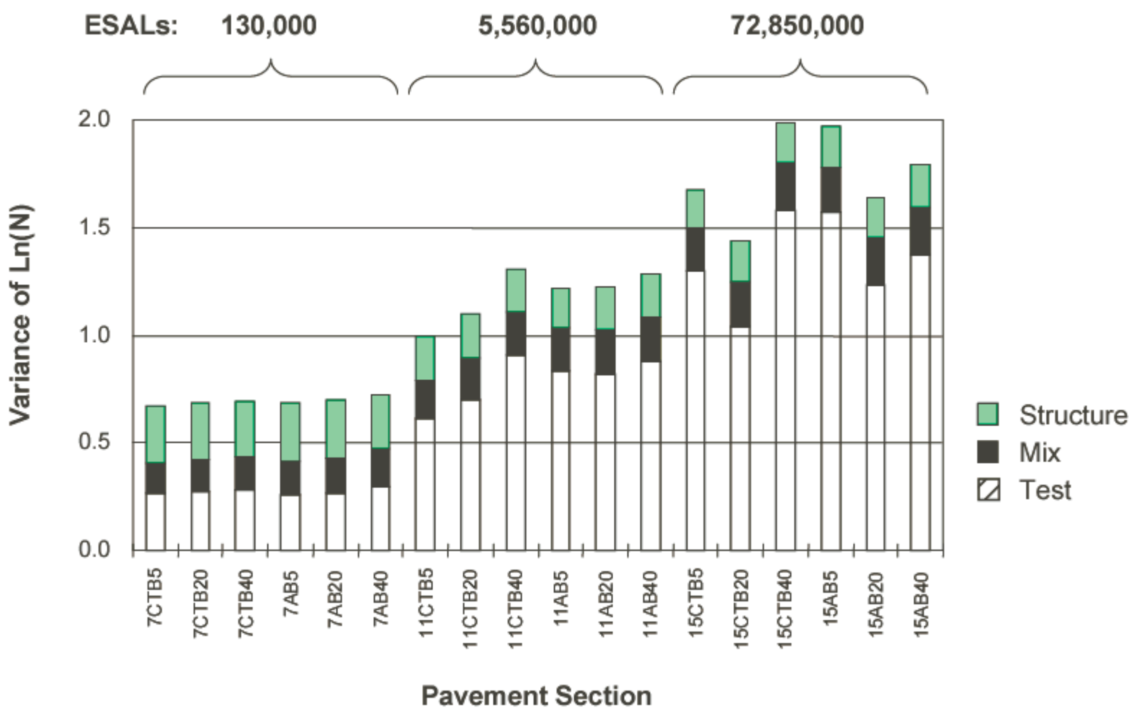


Figure 44. Contributors to variance of Ln(N).

ACCELERATED PAVEMENT TESTING

As discussed earlier, engineers have utilized accelerated pavement tests as well as observations of performance of test roads and in-service pavements to calibrate, validate, and modify, if required, their procedures for the design of new and rehabilitated pavements. Examples included work developed by the USACE-WES and the TRL in the UK.

In the United States, a number of test roads have been conducted under the aegis of the Transportation Research Board of the National Research Council, and the FHWA. Three that are of particular import are the WASHO Road Test (*5*), the AASHO Road Test (*6*), and WesTrack (*132*). The results of the AASHO Road Test have had a significant impact on pavement design, as already discussed earlier.

While it was noted earlier that the Benkelman Beam, as a pavement deflection measuring tool, had its inception at the WASHO Road Test, this test road provided other important contributions to asphalt pavement engineering as well. Two of these were (*133*):

- Thicker layers of asphalt concrete improved pavement performance; and
- Paved shoulders (as compared to unpaved shoulders) improved pavement performance in the lane adjacent to the shoulder and, in particular, the outer wheel path in this lane.

WesTrack, a federally-funded multi-million dollar hot-mix asphalt (HMA) located near Reno, Nevada, was completed in 2000. Its purpose was to further the development of performance-related specifications (PRS) technology and to provide field verification of the SHRP-developed Superpave asphalt mix design procedure. A series of asphalt mixes covering a range of aggregate gradations, asphalt contents, and degrees of compaction (35 sections in total) were evaluated (*132*).

Results from this test road provided important information on the effects of mix and construction variables on pavement performance. It has contributed to the concept of combining mix design and pavement design described in the previous sections and to the formulation of pay factors for use in performance-related specifications for hot mix asphalt construction (*91*).

Accelerated pavement testing (APT) has been an integral part of the development of the CBR procedure by the USACE. When this procedure was first developed, results of accelerated load tests at thirteen different locations were instrumental in establishing the initial thickness versus CBR relationships (*2*). The multiple wheel heavy gear load (MWHGL) tests of the 1960’s played a significant role in establishing the methodology of the current procedure for airfield pavements (*3*).

The concept of subjecting in-service pavements to accelerated loading was successfully promulgated by the Council of Scientific and Industrial Research (CSIR) of South Africa beginning in the 1970’s. They developed a series of Heavy Vehicle Simulator (HVS) units which could be used to test in-service pavements or specially developed test sections. The results of the tests using the HVS equipment together with laboratory test programs and pavement analyses have been successful in improving pavement technology including pavement construction practices. Early acceptance of analytically-based design in South Africa was assisted by the results of the HVS test program (*134*).

The success of the APT program in South Africa led to the development of an accelerated test unit in Australia termed the Accelerated Loading Facility (ALF) and a successful test program using the equipment to evaluate a range of paving materials (*135*).

The ALF technology was adopted by the FHWA and is currently being used at the FHWA Turner Fairbank Highway Research Center (TFHRC) to validate the binder specifications developed during the SHRP endeavor to control both permanent deformation and fatigue cracking in asphalt pavements (*136, 137*).

With the demonstrated success of APT in South Africa and Australia, a number of agencies both in the U.S. and abroad have developed their own capabilities. Currently, the states of Louisiana, Texas, California, Kansas, and Indiana have APT units. Reference (*135*) provides an excellent summary of the state of APT throughout the world as of 1996.

APT, to be effective, must be used in conjunction with both pavement analyses of the type described earlier and laboratory test programs. An example of this is illustrated in Figures 45 and 46 (*140*). Figure 45 shows one of the HVS units purchased by Caltrans (*139*). It was used to validate mix designs used in the rehabilitation of the I-710 freeway in Long Beach, CA (*140*). Results of the HVS test on the mix proposed for the surface course for the pavement is illustrated in Figure 46. The mix, containing a modified binder, was subjected to approximately 200,000 uni-directional repetitions at a pavement temperature of 50ºC at a 50-mm depth with the HVS shown in Figure 45. The mix design had been developed using the SHRP-developed shear test with anticipated rutting not to exceed about 12.5 mm after 660,000 repetitions (*140*). The results would appear to provide confidence in the mix design. Results of the three HVS tests shown are for mixes designed according to current California DOT practices (*139*) tested under the same conditions.

Figure 45. Heavy Vehicle Simulator.

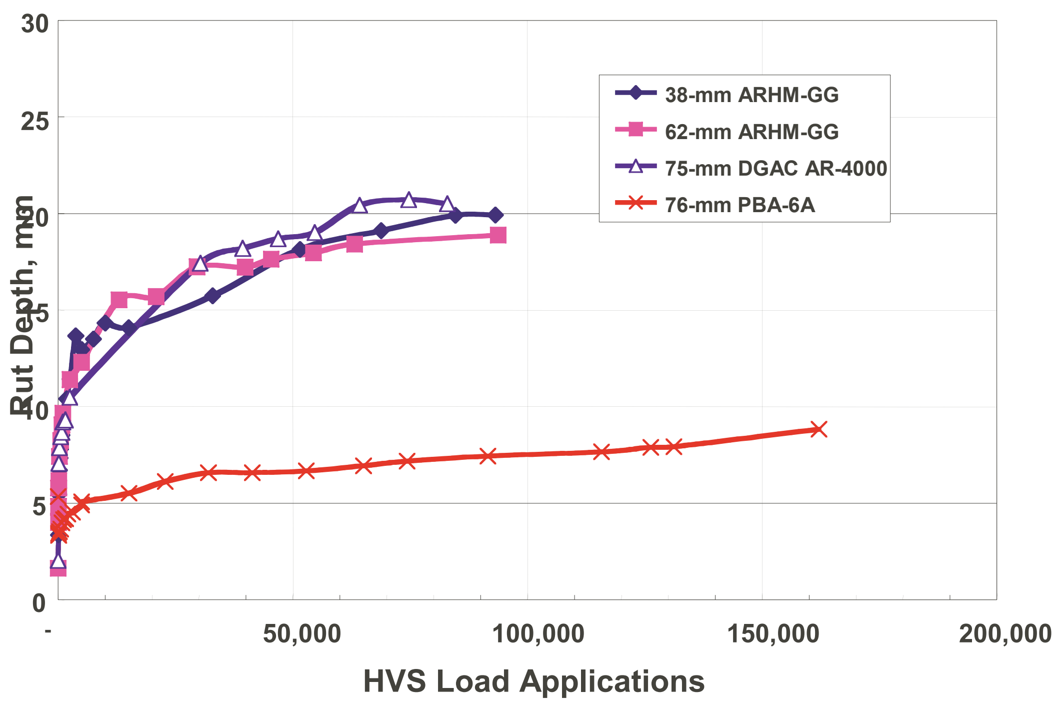


Figure 46. Rut depth versus HVS load applications with 40-kN load on dual tires at 50ºC.

PAVEMENT MANAGEMENT SYSTEMS

While engineers have managed their pavement systems in modern times, performing periodic maintenance and rehabilitation operations when deemed appropriate and tempered by available funds, it was not until the late 1960s that the concept of pavement management systems was introduced. In the United States, this occurred under the aegis of the National Cooperative Highway Research Program (NCHRP) Project 1-10 with F.N. Finn, Figure 7, and W.R. Hudson, Figure 47, as the principal researchers (*140*). At about the same time, in Canada, R.C.G. Haas began working in this area, Figure 48 (*141*).

Figure 47. W.R. Hudson.

Figure 48. R.C.G. Haas

The first state DOT in the United States to embrace this concept was the State of Washington Highway Department. The State Materials Engineer, R. LeClerc, working with F.N. Finn and his associates, developed a system to manage the entire state highway network (*142, 143*). This included:

1. the introduction of condition surveys to be done on a systematic and continuous basis, and which included measures of surface distress, ride quality, and skid resistance,
2. development of pavement performance relationships,
3. establishment of levels of performance requiring maintenance and rehabilitation, and
4. lifecycle cost analysis to permit effective use of existing funds.

At about the same time in Canada, R.C.G. Haas started a similar type of program for the Canadian Provinces (*141*).

The first truly network-level system was developed for the Arizona Highway Department in the 1970s by F.N. Finn and his associates working with G. Morris, the Arizona Research Engineer (*144*). The system was formulated to:

1. estimate costs to bring the network to and maintain it at some desired level of serviceability, or
2. in the face of budget constrains, estimate resulting serviceability's associated with the specific budget.

To accomplish this, an optimization model was utilized which based the formulation of the problem as a Markovian decision process and its conversion into a linear program, a remarkable advance for the time (*145*).

Since that time, pavement management systems have become a regular part of the activities of departments responsible for street and highway systems internationally. These systems provide the basis for maintaining entire networks at established levels of service commensurate with the designated functions of each of the segments of the system, e.g., interstate versus secondary routes.

In addition to the role of resource allocation, pavement management systems permit linking performance to the following databases:

1. initial design: materials and pavement sections;
2. construction, including as-built pavement sections, QC/QA data;
3. traffic data; and
4. environmental data.

Reference (*146*) provides an example of such a system in which the database with properties of mixes developed by the Superpave mix design have been linked to the pavement sections in the pavement management system so that their performance can be evaluated.

This approach permits development of improved design and rehabilitation methodology and improved construction procedures. In effect, by this linking, each highway network becomes a long-term pavement performance project. As AASHTO moves ahead with the new pavement design methodology this linkage will be extremely important for its validation.

SOME OBSERVATIONS

From the material presented, it is apparent that considerable progress has been made in the methodology for the design and construction of long lasting pavements. This has resulted from the cooperation of many people at the international level, with those involved freely sharing their knowledge and experience to advance the field of pavement engineering. Sufficient evidence has been presented, I believe, to conclude that technology is available to support the two premises stated at the outset, i.e.:

1. *Long lasting asphalt pavement can be designed using the developments in mechanistic-empirical (analytically-based) design over the past 40-plus years.*
2. *While long lasting asphalt pavements can be designed, careful attention to good construction practices is required to insure the anticipated design performance.*

For these premises to be implemented in an effective manner, *well educated people at all levels must be available.* This requires up-to-date education and training for engineers, technicians, and construction and maintenance personnel. As demonstrated herein, *Pavement Engineering is a "high-tech" profession!* In the U.S. at least, there is a growing concern that in the majority of Civil Engineering programs in Universities and Colleges, this premise is not understood by many current faculty and that education in Pavement Engineering at both the undergraduate and graduate levels has been diminishing.

A strongly recommended solution is to educate faculty as well as students. In the period 1956 to about 1965, the Asphalt Institute supported summer programs at a number of major Universities to educate Faculty in both asphalt pavement technology and pavement design and rehabilitation. Currently, an excellent example is the National Center for Asphalt Technology (NCAT) Instructor Training Course. More programs of this type, supported by Industry, can contribute to alleviating the faculty problem.

With the advent of web-based educational developments, *self-managed learning* programs like those developed by Professors J.P. Mahoney and S. Muench at the University of Washington provide up-to-date information for people new to the field as well as for more experienced personnel. An excellent example is the "Guide for Hot Mix Pavements,"(*147*) which can be viewed at the following website:

http://hotmix.ce.washington.edu/wsdot_web/WSDOT_intro.htm

Programs like this can be used as prerequisites for other forms of education and training. Such activities include both short courses developed through Technology Transfer (T^2) Centers; certifications, e.g., for construction personnel, and testing technicians.

In conclusion, while there are still gaps in our knowledge, the pavement engineering community has made great strides since 1962. It is the responsibility of those in the pavements area to apply this develop with good engineering judgment because:

"Nothing will be attempted if all possible objections must first be overcome."
(Anonymous)

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