Considerations in Airport Pavement Management

C. L. Monismith, Department of Civil Engineering and Institute of Transportation Studies, University of California, Berkeley

The state of the art of airport pavement management systems is discussed. Flow diagrams of several complementary management subsystems are presented. The input of information needed for these subsystems includes traffic (load), environmental effects, available materials and layer thicknesses, construction effects, and maintenance and rehabilitation considerations. A physical description of a pavement system (i.e., materials characteristics and dimensions of the various layers—such as those of the procedures developed at Shell, the Asphalt Institute, and the U.S. Army Engineer Waterways Experiment Station) serves as an introduction to a discussion of the various pavement models available (i.e., the experimental—such as the California bearing ratio—and the mathematical—such as multilayer elastic and viscoelastic systems). The forms of distress (fatigue, distortion, and fracture) are analyzed, and the structural design procedures (conventional and based on elastic layer theory) that can be used to minimize it are evaluated. The relations among performance evaluation, pavement overlay design, and maintenance management are considered.

The design and rehabilitation of airport pavement systems can be considered within a general framework such as that shown in Figure 1 (1). This paper will briefly outline the various elements of the process and provide a perspective for viewing the results of the research that is summarized in this Special Report.

There are no workable systems that completely describe the airport pavement management process shown in Figure 1, but there are a number of subsystems that can be used in conjunction with one another to provide engineers concerned with pavement design and rehabilitation a framework within which to make reasonable decisions. It is these subsystems that will be addressed here.

In Figure 1, the management process includes design, maintenance, and rehabilitation. In this discussion, maintenance is considered to include crack filling, patching, and minor repairing, but not such tasks as keeping runways and taxiways free of debris or grass cut. Rehabilitation includes the reconstruction and overlays necessitated by reductions in ride quality (e.g., increased roughness that influences the aircraft, reduction in skid resistance, or increased tendency toward hydroplaning).

INPUT

In the pavement system shown in Figure 1, various input data are required for both initial design and subsequent rehabilitation. The general categories of input include (a) traffic (load), (b) environmental effects, (c) available materials and layer thicknesses, (d) construction effects, and (e) maintenance and rehabilitation considerations.

Traffic (Load)

The types of traffic information required for reasonable estimates of performance are summarized below:

1. Gear configurations of representative aircraft using the facility,
2. Contact (or tire) pressures of representative aircraft,
3. Aircraft masses as affected by length of flight and takeoff and landing operations,
4. Daily and seasonal variations in aircraft movements,
5. Lateral distributions of loads on taxiways and runways and longitudinal distribution of loads on runways,
6. Aircraft velocities, and
7. Special loading considerations (e.g., braking and turning movements).

Some simplifications of these types can be made. Deacon (2) and Witczak (3) have suggested the use of equivalent loads; Witczak, for example, has defined the repetitions of all aircraft in terms of the repetitions of a fully loaded DC-8-63F.

The effects of the lateral distribution of aircraft gears on both taxiways and runways must be included to ensure economical designs; Witczak (4) has shown how this might be accomplished. HoSang, in a paper in this Special Report, has developed data that permits the use of simplified procedures to include these effects.

That turning effects can be important, particularly in high-speed exit-taxiway designs, has been shown by Witczak in an analysis of pavement sections at Baltimore-Washington International Airport (5), and the results of studies by Ledbetter, discussed in a paper in this Special Report, confirm these findings. Ledbetter's studies also indicate that the current practice of building taxiways and runways with end sections that are thicker than their interior portions is reasonable.

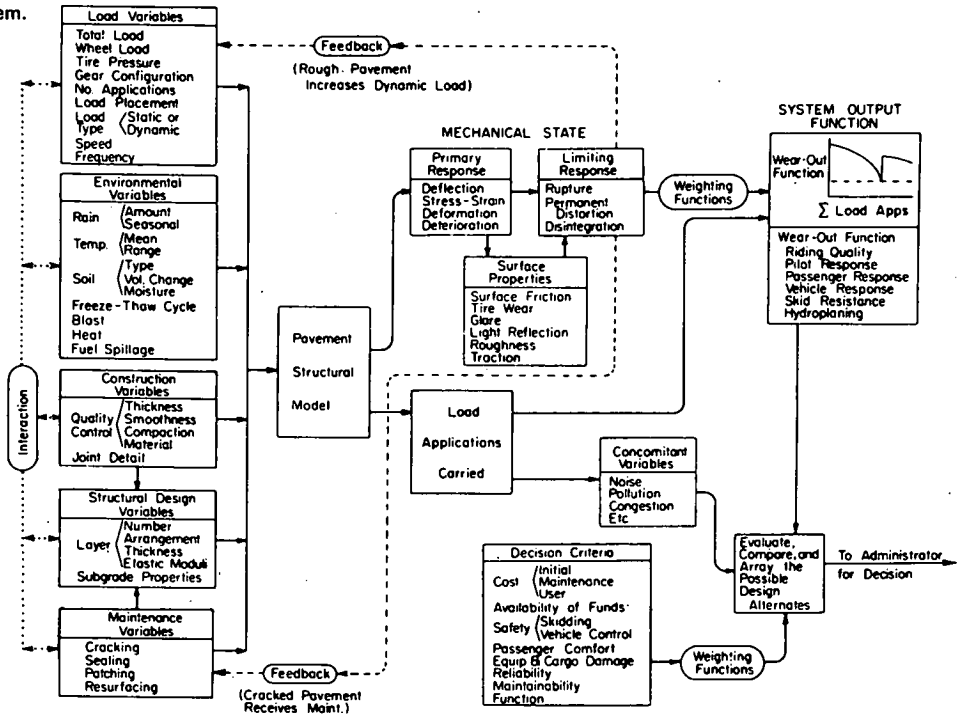
The effects of braking forces should also be evaluated; their inclusion may require thicker layers of asphalt-bound materials over untreated aggregate bases to preclude slippage failures.

Environmental Effects

The response of a pavement is affected by the environmental conditions such as temperature and moisture in which it is situated. The design considerations associated with these factors are summarized below.

Environmental Factor	Design Consideration
Temperature	Material stiffness Thermal stress Frost heave
Moisture	Material stiffness Warping stresses Frost heave Suction Volume changes

Figure 1. Airport pavement system.



Temperature

The response of asphalt-treated materials is temperature dependent, which influences their behavior in pavement sections. Temperature changes cause thermal stresses in both asphalt concrete (AC) and portland cement concrete (PCC); one of the necessary and sufficient conditions for frost heave is a prolonged period of freezing. Updated procedures for frost design are given in the summary presented by Johnson in this Special Report.

The distribution of temperatures in pavement layers can be estimated from weather data (i.e., average air temperatures and their daily ranges, average wind velocities, solar insolation, and sky cover) by using various solutions of the heat-conduction equation (6, 7):

$$\partial^2 \Phi/\partial z^2 = (\gamma c/k)(\partial \Phi/\partial t) \quad (1)$$

where

- Φ = temperature field,
- z = depth below pavement surface,
- c = specific heat of material,
- k = thermal conductivity,
- γ = density of material, and
- t = temperature.

[For AC, c is approximately 800 J/kg·K (0.2 Btu/lb·°F), and k is approximately 10.12 W/m·K (0.7 Btu·ft/h·ft²·°F).]

Moisture

One of the most important environmental effects is that of water, both because it affects the response of materials in pavement sections to load and because it may cause undesirable volume changes (e.g., frost heave or the expansion of clay).

For design purposes, the effect of water can be evaluated by measuring the properties of untreated materials in the saturated condition [e.g., the soaked California bearing ratio (CBR)] (8). In some instances, however, such procedures may actually be evaluating soil conditions that are not representative of those in the field (9), and thus it is desirable to have alternative procedures that provide estimates of expected in situ moisture conditions and an indication of how these conditions might develop, e.g., measures of the rate of increase (or decrease) in water content of the subgrade soil with time.

Where little or no freezing of the subgrade occurs, the soil-moisture suction may provide a useful (and practical) approach to the estimation of equilibrium moisture conditions in fine-grained soils underlying thick AC sections or sections containing other treated layers resting directly on the subgrade. McKeen's paper in this Special Report addresses this point.

Richard's (10) suggestions for suction estimates appear to be the most useful available at this time: For areas having shallow water tables [e.g., 6 m (20 ft) in clay, 3 m (10 ft) in sandy clays and silts, and 1 m (3 ft) in sand], regardless of climate, the equilibrium suction profile can be estimated from

$$\text{matrix suction}_z = \text{depth to ground water table} - z \quad (2)$$

where z = depth measured from bottom of impervious surface at which suction is desired. The total suction is equal to the sum of the matrix (or soil-water) suction and the osmotic suction. In the absence of dissolved salts, the osmotic suction is zero; for uniform salt concentrations, the osmotic suction can be neglected. Under these circumstances, therefore, the total suction can be considered equal to the matrix suction. For areas having deep water tables, the suction profile is controlled by the moisture balance between rainfall and evapotranspiration. In areas where there is no permanent surface desiccation, the profile can be expressed approximately by the relation

$$\text{Suction at depth } z = \text{suction at depth greater than depth of seasonal variation} + z_0 - z \quad (3)$$

where z_0 = depth greater than depth of seasonal variation

and z = depth at which suction is desired.

Alternatively, the total suction (and therefore the matrix suction) under covered areas can be related approximately to climate by Thornthwaite's moisture index (I) (11).

$$I = (100D - 60d)/E_p \quad (4)$$

where

D = soil drainage,
d = soil moisture deficit, and
E_p = potential evapotranspiration.

Local environmental factors can cause large departures from this relation; however, it may be useful for preliminary estimates of suction.

For locations where freezing and thawing can occur, updated procedures for predicting the depth of frost penetration and the effect of subsequent thaw are discussed by Johnson in a paper in this Special Report. Studies by Bergan (12) provide a way of defining the stiffness characteristics of fine-grained subgrade soils for use in design for these conditions.

Moisture differentials contribute to warping stresses in concrete slabs. The available evidence suggests that the effect of moisture-induced stresses is to reduce the magnitude of the warping stresses caused by temperature differentials. Pretorius (7) has shown that humidity gradients can lead to large tensile stresses in cement-treated bases. For a particular situation, therefore, the designer must satisfy himself that these factors are at least considered.

Available Materials and Layer Thicknesses

A variety of materials are available for the construction of structural pavement sections that have different surface requirements (e.g., improved skid resistance) and different maintenance characteristics. The proper selection of the particular combination of materials is dependent on economics; it is the role of the engineer to construct a pavement system that has the minimum cost for some prescribed time period by considering not only the initial design but also maintenance and rehabilitation.

Figure 2 illustrates the range of materials that can be used in a pavement section and the layers in which they are arranged. Figure 2 does not show the materials used for maintenance; these include (a) asphalts for spray applications (e.g., fog seals) and crack filling, (b) joint fillers, (c) rejuvenating agents for asphalt surfaces, and (d) special protective coatings for both AC and PCC layers.

Information about continuously reinforced concrete (CRC), fibrous concrete, and prestressed concrete and about porous asphalt friction courses is discussed by Parker and by White and Duggan respectively in papers in this Special Report.

The thicknesses of the layers are also variables, in part controlled by structural considerations, but there are some minimum thicknesses that should be maintained. Normally, layers of untreated materials should be placed in minimum thicknesses of 150 mm (6 in). Treated layers (with the exception of AC) should be placed with minimum thicknesses of 100 mm (4 in) and preferably of 150 mm. Portland cement concrete layers should not be less than 200 mm (8 in) thick for heavy-duty pavements, while AC layers should not be less than 50 mm (2 in) thick (when AC is used for overlays on PCC pavements, a minimum thickness of 100 mm is recommended).

Construction Effects

The inherent variability attendant in the construction process must also be included. Kennedy and others (13) have summarized some of the available literature. They provide guidelines so that the designer can at least qualitatively consider such variability as a part of the design and rehabilitation process. In this Special Report, R. Brown discusses material variability and E. Brown discusses statistical quality-control requirements.

Maintenance and Rehabilitation Considerations

As shown in Figure 1, both maintenance and rehabilitation must be considered in the pavement-management process.

Maintenance will affect pavement performance. It is possible, for example, that the sealing of cracks may prevent water infiltration to underlying layers and thereby reduce the potential for pavement deterioration due to reduced stiffnesses in these layers.

For AC pavements, the placement of overlays before actual cracking (programmed stage construction) may provide longer service lives than the placement of overlays after cracking is visible on the pavement surface. This planning must, however, be incorporated in the overall pavement-management process.

PHYSICAL DESCRIPTION OF PAVEMENT SYSTEM

To design new structures and to estimate the load-carrying capacity of or plan rehabilitation for existing structures require definitions of the materials characteristics and the dimensions of the various layers that are being considered for or actually comprise a pavement structure.

Materials Characteristics

The selection of appropriate materials characteristics depends on the design or rehabilitation methodology that is being used. In this section, a brief summary of the characteristics associated with some of the design procedures that have been used for a number of years and of the characteristics required for a few of the new procedures will be presented.

Characteristics in Current Use

For flexible pavements, the most widely used design procedure is that in which the California bearing ratio (CBR) test defines the requisite load-carrying characteristics of the pavement components (8). For fine-grained soils, the CBR is usually determined after the material has been soaked for 4 d. If it can be demonstrated that the subgrade will not become saturated, the test can be conducted at the expected condition in situ. Figure 3 illustrates the effects of compaction conditions on the soaked CBR of a lean clay soil. Analysis of data in this form assists in selecting the field-compaction conditions to ensure a reasonable strength after soaking.

For untreated granular materials and treated fine-grained soils and granular materials, CBR values generally are assigned on bases of other test characteristics, such as gradation and plasticity.

In the design of PCC pavements, it is necessary to define the modulus of subgrade reaction (k) for the layered structure on which the slab is to be placed (14). This modulus can be determined by plate-load tests (e.g., ASTM D1195-64 or D1196-64) or estimated from

other soil parameters (14).

Stiffness (Modulus) Characteristics

In the Shell (15) and Asphalt Institute (AI) (16) methods of airfield-pavement design, the stiffness or elastic characteristics of the materials comprising the pavement sections are required; similarly, in the procedures being developed at the Waterways Experiment Station (WES) for the Federal Aviation Administration (FAA), stiffness values have been used. This methodology is discussed by Barker in a paper in this Special Report.

Stiffness characteristics can be used to evaluate the performances of existing pavements and in the design of overlays (5, 17, 18), as well as in the design of new pavements.

Modulus values for PCC are necessary for the widely used analysis procedures that estimate the stresses in the concrete slabs that result from loading and environmental influences.

Untreated Soils

For untreated materials, a measure of stiffness termed the resilient modulus (M_R) and determined from repeated-load triaxial compression tests is suitable for use in the various elastic analyses that examine the effects of moving wheel loads on pavement structures. This modulus is defined by the following relation

$$M_R = \text{repeated axial stress/recoverable axial strain} \quad (5)$$

For fine-grained (cohesive) soils, M_R is dependent on the applied axial stress; i.e., $M_R = f(\sigma_{axial})$, which emphasizes that this modulus should be determined for the range of stresses that can be expected to occur in situ. The M_R is also dependent on the water content or suction (Figure 4).

For untreated granular (cohesionless) materials, the modulus is also dependent on stress; for these materials the form of the relation is

$$M_R = K\Theta^n \quad (6)$$

where $\Theta = \sigma_1 + \sigma_2 + \sigma_3$ (sum of principal stresses) and K = constant. Figure 5 illustrates data obtained for an untreated base material used in a state highway pavement in California. The relation is also dependent on water content (or degree of saturation), with the modulus decreasing with increase in water content.

Poisson's ratio for fine-grained soils is also dependent on suction, ranging from about 0.3 at a high suction to 0.5 at zero suction. For granular materials, Poisson's ratio is somewhat dependent on the ratio of σ_1 to σ_3.

Asphalt-Bound Materials

The stiffness characteristics of asphalt-bound materials using asphalt cements can be defined by the relation

$$S(t,T) = \sigma/\epsilon \quad (7)$$

where

s(t, T) = mixture stiffness at a particular time of loading and temperature,
σ = stress, and
ϵ = strain.

The stiffness can be measured by one of the following procedures (a) creep, (b) vibration, or (c) repeated axial or flexural loading [triaxial compression, third-point flexure, or diametral (split tension)]. Alternatively, stiffness can be estimated from knowledge of (a) the penetration of asphalt cement at 25°C (77°F), (b) the temperature corresponding to a penetration value of 800, (c) the volume concentration of the aggregate in the mix, or (d) the air-void content of the mix by using the procedure developed at Shell (19). Figure 6 shows a comparison of measured and estimated stiffness values.

For asphalt-emulsion-treated materials in the partially cured state, an expression similar to Equation 6 is applicable.

Schmidt and Graf (20) have presented data illustrating the effects of water on mix stiffness, in which stiffness was measured with the diametral device (Figure 7). They have also shown that asphalt hardness and the addition of slurry lime improve the retention of mixture stiffness in the presence of water.

Poisson's ratio is dependent on both time of loading and temperature and ranges from 0.3 at low temperatures and short loading times to 0.5 at high temperatures and long loading times.

Lime- and Cement-Stabilized Materials

Stiffness data for lime-treated materials have been given

Figure 2. Materials for pavement sections.

SPECIAL SURFACINGS - E.G., POROUS FRICTION COURSE, SPECIAL SEAL COAT		
	ASPHALT CONCRETE	
	PORTLAND CEMENT CONCRETE	UNREINFORCED CONTINUOUSLY REINFORCED PRESTRESSED FIBROUS
BASE:	ASPHALT TREATED (ASPHALT CONCRETE) CEMENT TREATED LIME-FLY ASH TREATED UNTREATED AGGREGATE MEMBRANE ENCAPSULATED SOIL LAYER	
SUBBASE: (ONE OR MORE)	TREATED INCLUDING ASPHALT, PORTLAND CEMENT, LIME, LIME-FLYASH UNTREATED AGGREGATE INSULATING LAYER	
SUBGRADE:	COMPACTED: TREATED (E.G., WITH LIME) UNTREATED	
	NATURAL	

Figure 3. Relation between initial composition and soaked CBR for samples of lean clay.

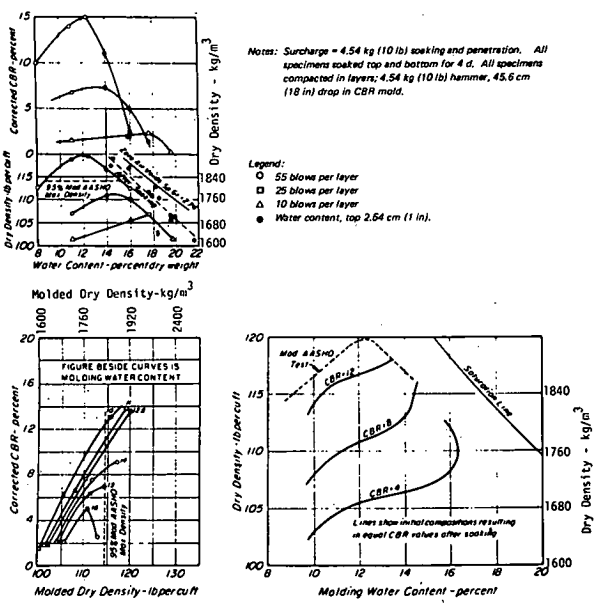


Figure 4. Relations among water content, dry density, and resilient modulus for subgrade soil.

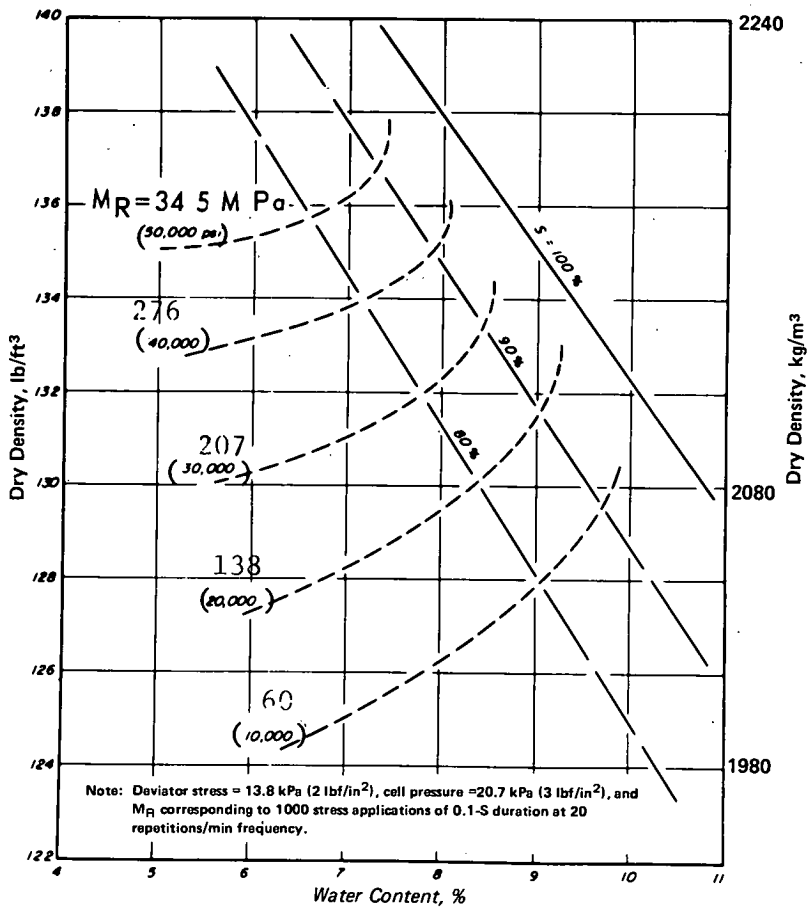
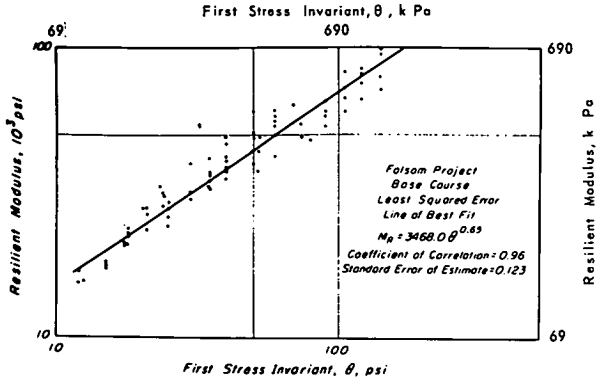


Figure 5. Individual test results—modulus versus sum of principal stresses—for base-course aggregate.



by Thompson (21) and by Mitchell and others (22). Thompson's data indicate that the modulus of these materials is directly related to the unconfined compressive strength and that modular ratios for lime-treated to untreated soils are in the range of 3 to 25. The results of both studies indicate that after a reasonable curing period, lime-treated materials have essentially elastic responses.

Stiffness data for cement-stabilized materials have been summarized by Mitchell (23). Stiffness values may range from 70 MPa (10 000 lbf/in²) to about 28 GPa (4 000 000 lbf/in²) depending on type of soil, treatment level, curing time, water content, and test conditions. Treated-fine-grained soils have stiffness values near the lower end of the range whereas granular materials have higher values. For stress levels in the working range, Poisson's ratio varies from 0.1 to 0.35, depending on the same conditions that affect stiffness.

Portland Cement Concrete

In the existing design procedures, a modulus value of 28 GPa and a Poisson's ratio of 0.15 are used. Packard (14) has shown that changes from these values have only a slight effect on design thicknesses.

Thermal Characteristics

To determine temperatures in pavement structures requires knowledge of the specific heat (capacity) (c) and the thermal conductivity (k). The values for asphalt mixtures are given above. For cement-stabilized soils, k ranges from 0.05 to 0.12 W/m·K (0.3 to 0.7 Btu·ft/h·ft²·°F); an average value of 0.08 W/m·K (0.5 Btu·ft/h·ft²·°F) appears reasonable for use. The specific heat of cement-stabilized materials is about 800 J/kg·K (0.2 Btu/lb·°F)—i.e., the same as that of AC. Both c and k for PCC are approximately the same as for AC. The thermal properties of untreated materials have been tabulated by Berg (24) and are summarized in this Special Report by Johnson.

Volume-Change Characteristics

The change in pavement surface elevation that will result from a volume change in an underlying clayey subgrade soil can be estimated if the water content (or void ratio) versus suction relation is measured for the subgrade

Figure 6. Stiffness modulus of asphalt concrete at various temperatures.

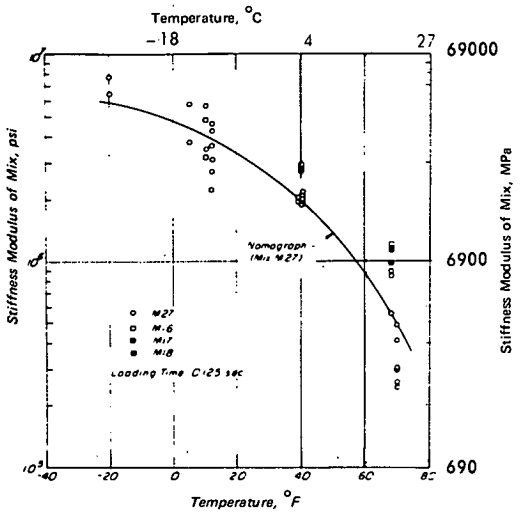
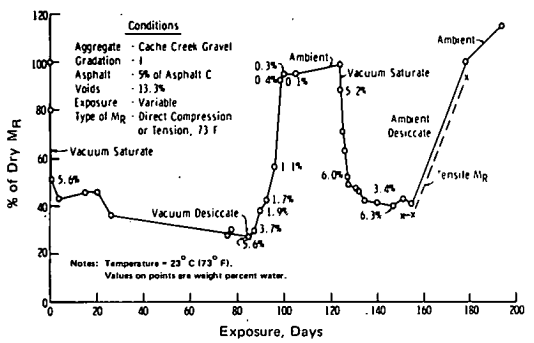


Figure 7. Effect of moisture on mixture stiffness (measured in diametral repeated-loading unit).



material. Various procedures are available for such determinations and are summarized in this Special Report by McKeen.

Materials Variability

The variability in materials characteristics must be included in the design and rehabilitation management process to ensure that the specific requirements for pavement serviceability will be obtained. Kennedy and others (13) have summarized the existing information on the variational characteristics of pavement materials for those currently used in pavement design. Table 1 (modified from their report) indicates the type of data required; in this case, variations in the values in resilient moduli for the subgrade soil.

Structural Section Geometry

In AC pavements, layer thicknesses are controlled by load considerations. However, construction requirements may also govern minimum thicknesses. Similarly, for PCC pavements, there are minimum recommended thicknesses for both the slab and the underlying layers. In addition, for PCC pavements requiring joints (although both CRC and prestressed concrete pavements require at least a few joints, these types of pavements are not included in this description), joint spacing will range from 3.8 to 7.6 m (12.5 to 25 ft) for longitudinal joints and from 3.8 to 15 m (12.5 to 50 ft) for transverse joints, depending on the construction equipment available, slab thickness, and whether or not reinforcing is used.

For CRC pavements, special attention must be given to the ends of the pavement (terminal design); these design considerations and joint design in prestressed concrete pavements are discussed by Parker in a paper in this Special Report.

PAVEMENT MODELS

The process of selecting the thickness of a pavement usually involves a procedure in which the designer uses the relations between load and a series of parameters representative of the various materials in the section. These relations can be based on either experimental studies or mathematical models and are usually modified by observations of pavement performance. Both types of relations are used in the thickness-selection process for design of new sections and for overlays.

Experimental

The most widely used procedure for airport pavements in this category is that using the CBR developed at WES. The expression relating CBR to thickness (25) is

$$T = \alpha_i[(ESWL/8.1\ CBR) - (A/\pi)]^{1/2} \quad (8)$$

where

α_i = load repetition factor,
ESWL = equivalent single wheel load, and
A = contact area.

α_i depends on the number of wheels for each main landing gear used to compute ESWL. ESWL is determined by assuming the pavement to behave as a homogeneous isotropic elastic solid and defined as the load on a single wheel having the same contact area and producing the same maximum deflection as the wheel assembly.

This relation has been developed from performance data, but also incorporates the use of a mathematical model—a homogeneous isotropic elastic solid—to estimate ESWL. As illustrated in Figure 8, materials variability is inherent in the design procedure represented by Equation 8.

Mathematical

For the design of concrete pavements, solutions originally developed by Westergaard (26) for a plate on a dense liquid have been used in both the Portland Cement Association (PCA) (14) and the Corps of Engineers (CE) procedures (27). More recently, the Shell (15) and Asphalt Institute (AI) (16) procedures for AC pavements have assumed the pavement to respond as a multilayer elastic solid with full continuity (friction) at each of the interfaces.

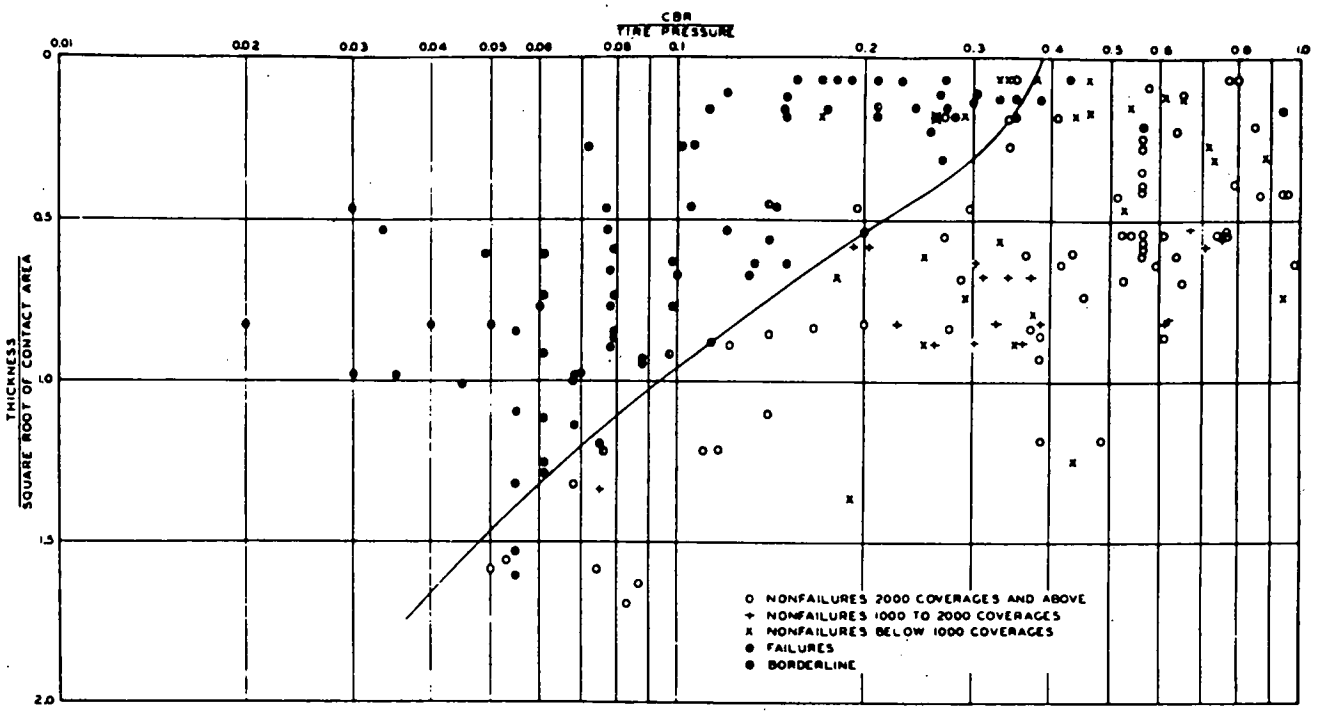
For AC pavements, the assumption that pavements respond as multilayer elastic systems is reasonable. There are a number of computer solutions available using integral-transform procedures to estimate stresses and deformations; these include CHEV 5L, BISAR, ELSYM, and GCP-1. In the BISAR and GCP-1 programs, both horizontal and vertical loads can be included, which permits the inclusion of braking forces at the pavement surface. When using such an analysis with pavements containing asphalt-bound layers, both time of loading and temperature effects on the stiffness of the materials must be considered. Portland cement concrete pavements have also been treated within the same general framework (28).

Table 1. Values of M_R for subgrade soils from airport pavements.

Airport	Location	Project Identification	No. of Tests	Deviator Stress (kPa) 55 Value	55 CV[a]	41 Value	41 CV[a]	69 Value	69 CV[a]
Palmdale	California	Runway 7-25	4	69 000	52	69 000	47	58 000	26
			10	165 600	26	207 000	29	262 200	34
O'Hare International	Chicago	Runway 9R-276	8	54 500	48	–	–	–	–
			3	43 500	21	60 000	12	69 000	13
		Runway 4R-226	4	39 300	17	48 000	5.3	58 000	16
Midway	Chicago	Runways 4R-22L and 13R-31L	4	16 600	10	27 600	7.9	37 300	6.6
Byrd International	Richmond	Taxiways S-4 and D and runway 2	3	59 300	8.3	82 800	20	75 900	18

Note: 1 kPa = 0.145 lbf/in^2.
[a]CV = coefficient of variation.

Figure 8. CBR criteria: results of Equation 8 versus behavior data.



An alternative to the use of multilayer elastic theory is the use of viscoelastic layer theory (29), but at this time, this approach does not provide the versatility embodied in the elastic layer theory. Viscoelastic analysis can be used, however, to examine the effects of long-term static vertical loads (e.g., in parking areas or at container-transfer facilities).

To determine the potential for low-temperature fracture in an asphalt-bound layer, the asphalt mixture can be represented as a viscoelastic slab (30), and other elastic solutions are also available (31). Other procedures have been developed for specific situations—e.g., the finite element method has been used to ascertain the stresses associated with traffic load (32, 33), and both finite element and finite difference methods have been used to estimate pavement temperatures (7, 34).

FORMS OF DISTRESS

As shown in Figure 1, the performance—i.e., the ability to carry out its intended function—of a pavement is directly related to distress; accordingly, estimates of the potential for distress and its effect on performance should be defined as a part of the design and rehabilitation process. Some of the types of distress that may occur in airport pavements are summarized in Tables 2 and 3. There are many forms, and it would seem appropriate, as in the highway field, to define those that are most widespread and to develop methods of preventing or minimizing their effects on performance for some prescribed period.

For asphalt type pavements, the three important modes are

1. Fracture from repeated loading,
2. Distortion (rutting) from repeated trafficking, and
3. Fracture from non-traffic-load-associated factors (e.g., temperature changes) and at times from braking stresses.

For PCC pavements, fracture from repeated loading and the effects of volume changes that result from various causes, particularly as they influence joint design to minimize the deleterious effects of fracture, are the two most important modes.

In this section, a brief summary of recent develop-

Table 2. Categories of distress for asphalt concrete pavements.

Mode of Distress[a]	General Cause	Specific Cause
Fracture or cracking	Traffic-load associated	Repeated loading (fatigue), which includes few as well as many repetitions of load applications
		Slippage, which results from braking stresses
		Reflection cracking, which may be accelerated by traffic loading
	Non-traffic-load associated	Thermal changes
		Moisture changes
		Shrinkage of underlying materials (i.e., reflection cracking)
Distortion	Traffic-load associated	Rutting, which results from repetitive loading
		Plastic flow or creep, which results from single or comparatively few excessive loads
	Non-traffic-load associated	Heave, which results from swelling clays or frost
		Consolidation settlement

[a]Disintegration, which is associated more with material than with structural design considerations, will not be considered in the initial design phase.

Table 3. Categories of distress for portland cement concrete pavements.

Mode of Distress	Manifestation	General Cause	Specific Cause	Examples of Distress
Fracture	Cracking	Traffic-load associated	Excessive loading	Longitudinal or corner cracking
			Repeated loading	Longitudinal, corner, or map cracking
		Non-traffic-load associated	Thermal changes	Transverse cracks
			Moisture changes	Transverse cracks
			Shrinkage	Transverse cracks
	Spalling	Traffic-load associated	Repeated loading	Dislodgment of portions of pavement at joint edges
		Non-traffic-load associated	Thermal changes	Dislodgment of portions of pavement at joint edges
			Moisture changes	Dislodgment of portions of pavement at joint edges
Distortion	Permanent deformation[a] (warping)	Traffic-load associated	Repeated loading	Changes in slab elevations from pumping[b] or densification of underlying materials
		Non-traffic-load associated	Expansive subgrade soil	Swell
			Frost-susceptible materials	Differential heave
	Faulting[c]	Traffic-load associated	Repeated loading	Differences in elevation between leave and approach slabs
		Non-traffic-load associated	Expansive subgrade soil	Differences in elevation between leave and approach slabs
			Frost-susceptible materials	Differences in elevation between leave and approach slabs
Disintegration	Scaling	Non-traffic-load associated	Action of deicing chemicals	Erosion of surface
			Freeze-thaw action	Erosion of surface
			Chemical reactivity of aggregate (alkali aggregate reaction)	Erosion of surface
	Raveling	Traffic-load associated	Repeated application of chains and studded tires	Erosion of surface
	D-cracking[c]	Non-traffic-load associated	Freeze-thaw action	Cracking to complete disintegration

[a]Blow-ups can lead to excessive displacements at joints (caused generally by high temperatures and lack of movement in the joint because of ingress of foreign materials).
[b]Pumping leads to longitudinal and transverse load-associated cracking that in turn results in permanent deformation in the slab.
[c]D-cracking could also be included in the fracture section because this type of distress first appears as cracks near slab corners.

ments is presented that permits estimation of the various forms of distress for both AC and PCC pavements. Specific distress modes and appropriate procedures to estimate these modes for both types of pavements are described.

These developments are included to provide background for some of the new methodologies that use them and to emphasize how they can be used with the existing methodology to make reasonable design and rehabilitation decisions.

Fatigue

A specific format with which to estimate the potential for fatigue cracking is shown in Figure 9. In this diagram, the material for which fatigue is to be considered is asphalt concrete, but any treated pavement layer can be analyzed in this manner. This section describes the various steps by which a structural pavement section can be analyzed according to this format. Christison (34) and Packard (14) describe in detail such approaches for AC and PCC respectively.

Materials Characterization

The magnitude of the tensile stress or strain repeatedly applied is a reasonable damage determinant for the various materials used in pavement sections. These stresses and strains can be estimated by using one of the methods of analysis described above (e.g., multilayer elastic theory). All of these procedures require the stiffness characteristics of the various materials; methods for their estimation and expected ranges in value have been described above.

Structural Analysis

The tensile stresses or strains resulting from wheel loads can be estimated for AC by using multilayer elastic, viscoelastic, or finite element analyses. For PCC, tensile stresses can be estimated by using the assumption of a plate resting on a dense liquid or by multilayer elastic or finite element analyses.

The effect of distributions such as those shown by HoSang in a paper in this Special Report can be readily evaluated by using multilayer elastic theory; the efficacy of such an approach has already been demonstrated in the AI design procedure (16).

Fatigue Characteristics

For asphalt-bound materials, the available data indicate that fatigue response can be defined by relations of the following form (36, 37, 38):

Figure 9. Block diagram of fatigue subsystem.

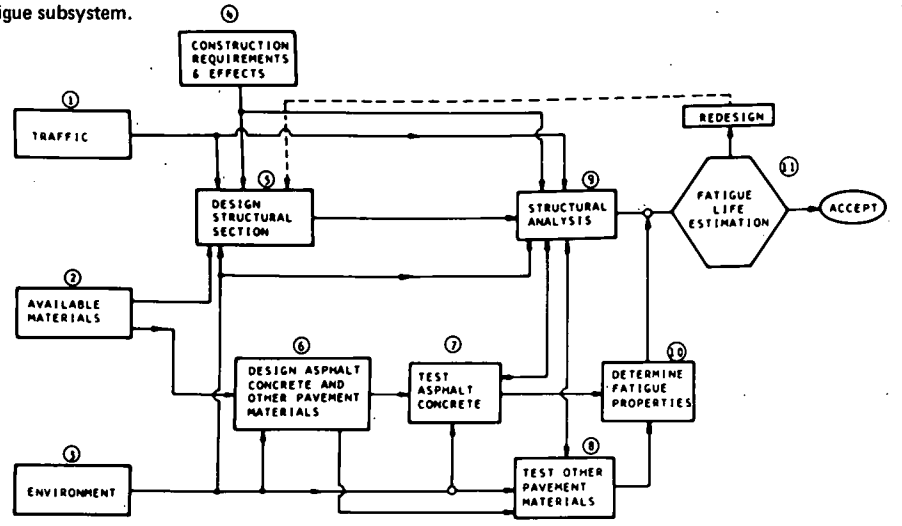


Figure 10. Initial bending strain versus applications to failure for controlled-stress fatigue tests on asphalt concrete.

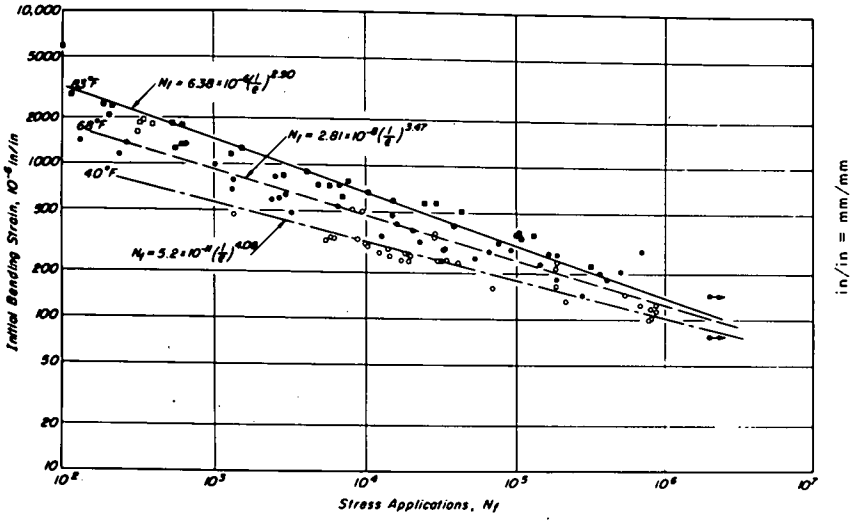
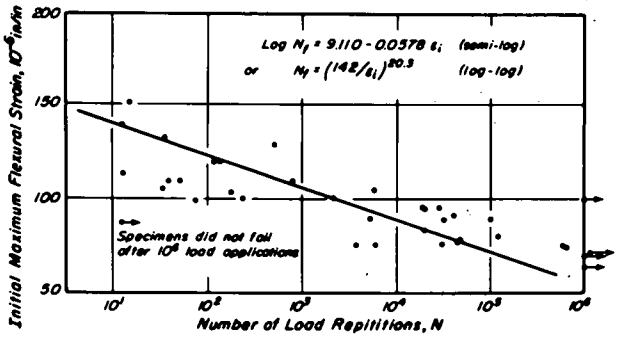


Figure 11. Fatigue life of flexural specimens.



$$N_f = A(1/\epsilon)^b \tag{9}$$

and

$$N_f = C(1/\sigma)^d \tag{10}$$

where

σ = tensile stress repeatedly applied,
ϵ = tensile strain repeatedly applied,
A, b, C, and d = material coefficients, and
N_f = number of applications to failure.

Typical data obtained from simple loading tests on a dense-graded asphalt concrete are shown in Figure 10. The relations drawn through the data are mean curves; there is also evidence that the distribution of fatigue lives at a particular stress level is log normal (37).

Some adjustment must be made in fatigue curves developed from laboratory data to include the effects of crack-propagation when the material is used in a pavement structure. Generally, this adjustment involves displacing the laboratory curves horizontally (i.e., on the N_f scale) by some factor [e.g., three to five (39)].

Asphalt content and degree of compaction (as measured by air-void content) affect fatigue response. These effects can be considered by adjusting the fatigue data determined for specific void and asphalt contents according to the relation (40)

$$N_f \sim [V_B/(V_v + V_B)]^x \tag{11}$$

where

V_B = asphalt volume,
V_v = air-void volume, and
x = coefficient (approximately 5).

For cement-treated materials also, the tensile strain repeatedly applied is a reasonable damage determinant (Figure 11). For soil cement, PCA has defined a generalized fatigue relation in terms of a critical radius of curvature (41) that has been validated by other investigators (7).

Figure 12. Comparison of coverage and fatigue strength relations.

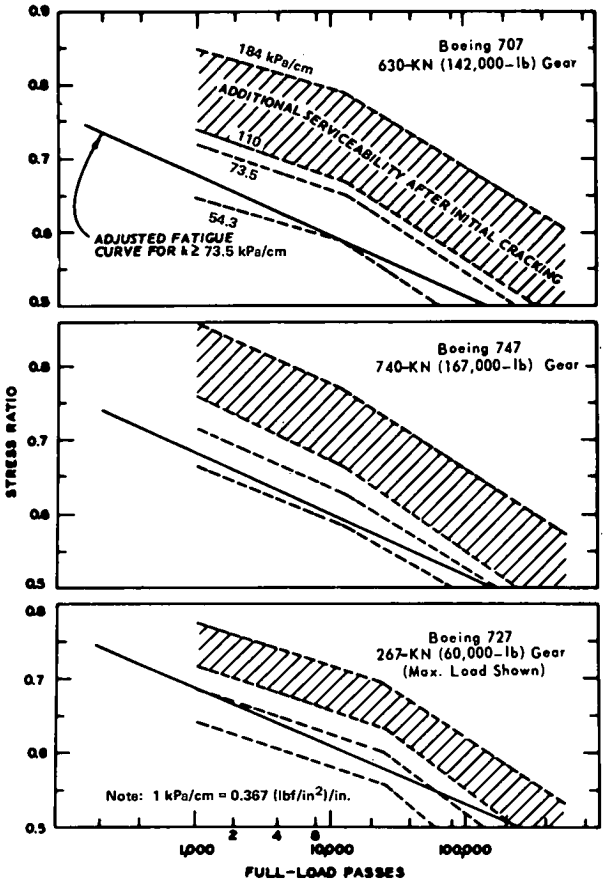
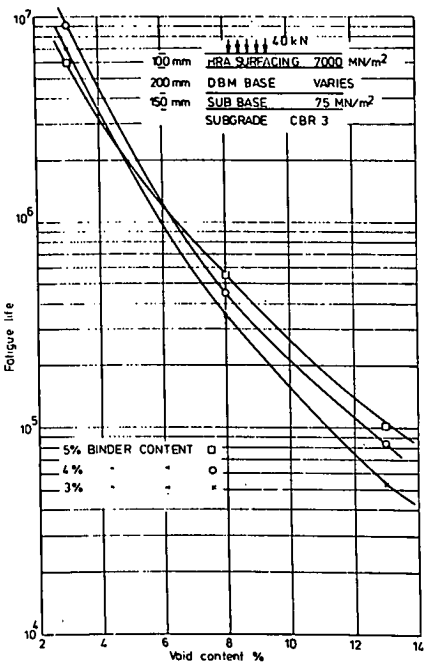


Figure 13. Effects of asphalt and void contents on fatigue behavior of dense bituminous macadam.



Recently, Raad and others (42) have suggested a technique that uses Griffith's theory to define the fatigue characteristics of cement-treated materials. Their criteria may provide generalized data that in turn may minimize the laboratory testing required for such materials.

Some fatigue data are available for lime-stabilized materials (43), although these data are not as extensive as those for cement-stabilized materials.

The PCA has suggested that the fatigue response of PCC can be defined in terms of the ratio of the applied stress to the modulus of rupture (at 28 d). When this ratio is 0.5 or less, the concrete is assumed to have an unlimited fatigue life. However, other investigators have suggested different approaches—e.g., in the CE procedure, a relation between a stress ratio termed the design factor (44) and the number of coverages is used. A comparison between this approach and that used by PCA is shown in Figure 12 (45). Other investigations have suggested an equation without an endurance limit (46).

Estimation of Fatigue Life

The estimation of the effects of a range in loading conditions requires a cumulative-damage hypothesis. One reasonable hypothesis is the linear summation of cycle ratios (35, 47):

$$\sum_{i=1}^{n} (n_i/N_i) = 1 \qquad (12)$$

where

n_i = number of applications at strain load i and
N_i = number of applications to cause failure in sample loading at strain level i.

This equation indicates that fatigue-life prediction for the range in loads anticipated becomes a determination of the time when this sum reaches unity. This equation is currently used in the AI design procedure for airfield pavements (7) and is suggested for use for concrete pavement design (14).

Fatigue relations such as those shown in Figure 10 are based on some chance of survival; e.g., the relations shown in Figure 10 are mean curves that represent a 50 percent probability of fatigue cracking. If a design is to

Figure 14. Effects of asphalt and void contents on fatigue behavior of hot-rolled asphalt surfacing.

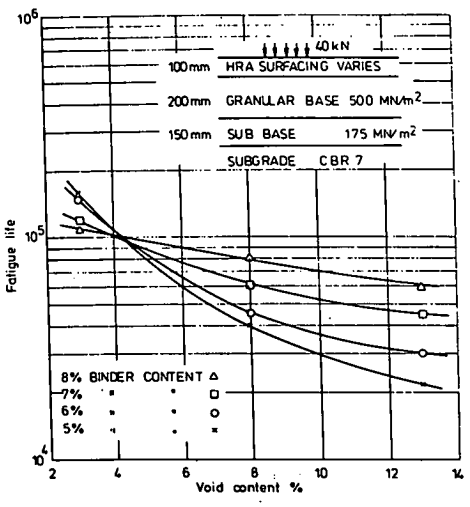
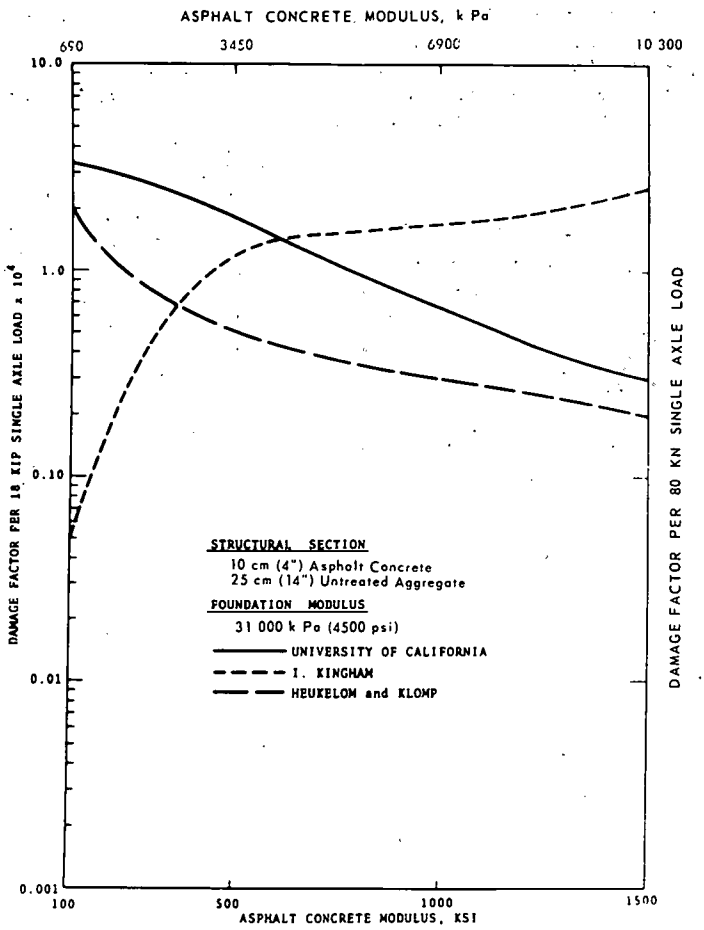


Figure 15. Effect of stiffness modulus on fatigue behavior of asphalt concrete.



be based on a lower possibility of cracking (e.g., 10 percent) it will be necessary to adjust the relation by assuming the distribution of fatigue lives to be log normal as noted above.

Other Considerations

Fatigue data can also be used to assess the effects of construction procedures on performance (e.g., the effect of field compaction on the fatigue performance of AC). This represents an important illustration of how available data together with theory can assist the engineer in assessing the consequences of design decisions and construction procedures.

Figure 13 (48) illustrates the results of an analysis of a pavement containing a dense bituminous-macadam base. Both the asphalt and the void contents of the macadam base were varied, and the stiffness was computed by the Shell procedure (19). The maximum tensile strains were computed at the underside of the asphalt-bound layer, and the nomograph recently developed by Pell and Cooper (40) was used to estimate the fatigue life. Pell notes that in this situation, the effect of void content is extremely important. However, the effect of asphalt content may be more important than shown because the procedure used to estimate stiffness may overestimate somewhat the effect of void content. The data do, however, stress the importance of good compaction.

Figure 14 (48) illustrates the results of an analysis of a pavement containing a comparatively thin asphalt-bound layer—hot-rolled asphalt. In this case, the effect of void content is not as significant because stiffness is not as important as for the pavement analyzed in Figure 13. The importance of higher asphalt contents at higher void contents is emphasized.

In general, these data indicate how one can analyze the effects of construction variables on performance. Moreover, they emphasize that such variables must be examined in the context of a particular pavement structure.

Finally, the importance of using proper criteria must be emphasized. Finn and others (49) have illustrated how criteria developed by different techniques can lead to different conclusions relative to the effects of mixture characteristics on fatigue performance of asphalt concrete. In the example shown in Figure 15, increased stiffness of AC leads to a reduction in fatigue life, but the other criteria lead to the opposite result.

Distortion

Distortion (or permanent deformation) can result from both traffic-load-associated and non-traffic-load-associated causes as seen in Table 2. A framework similar to Figure 9 to predict this mode of distress can be developed, but different materials-characterization procedures, methods of analysis, and distress criteria may be required.

Traffic-Load Associated

For asphalt-surfaced pavements, two approaches are available for the estimation of rutting from repeated traffic loading. In one method, the vertical compressive strain in the subgrade surface is limited to some tolerable amount associated with a specific number of load repetitions [e.g., the Shell method (15)]. Controlling the characteristics of the materials in the pavement section through materials design and proper construction

procedures (unit mass or relative compaction requirements) and using materials of adequate stiffness and sufficient thickness so that this strain level is not exceeded make it possible to ensure that permanent deformation is equal to or less than some prescribed amount. The second procedure involves an estimation of the actual amount of rutting that might occur by using appropriate materials-characterization information and an analysis procedure such as that described in the discussion of fatigue.

Limiting Subgrade Strain Criteria

The strain criteria developed by Witczak (3) are based in part on the analysis of field trials conducted at WES (50). These criteria, for a two-layer elastic pavement section in which E_1 = 690 MPa (1600 lbf/in^2) and Z_1 and Z_2 = 0.40 and 0.45 respectively, are summarized below (1 mm = 0.039 in).

No. of Load Applications	Material Compressive Strain on Subgrade (mm/mm) Asphalt Institute	Shell
1 000	0.001 92	–
10 000	0.001 68	–
100 000	0.001 52	–
1 000 000	0.001 46	0.000 90

To use these criteria, the pavement is analyzed as a layered elastic structure in the same manner as described above; the materials characteristics are the same as those required for the fatigue analysis.

Estimation of Rutting From Repeated Traffic Loading

A number of procedures are available for the estimation of rutting from repeated traffic loading, although to date none has been documented to the extent of the fatigue procedure. They can be categorized as

1. The use of an elastic layer theory to represent the pavement structure and materials characterization by either (a) repeated-load triaxial compression tests or (b) creep tests (at least for the asphalt-bound layers) or
2. The use of viscoelastic layer theory to represent the pavement structure and materials characterization by creep tests. [Morris and others (51) have indicated that this procedure requires additional study; hence, it will not be discussed in this paper.]

A number of investigators (52, 53, 54) have suggested that a pavement can be represented as a layered elastic system in the determination of the state of stress or strain resulting from a surface loading. The amount of rutting can then be estimated for some specified number of load repetitions by the use of an appropriate constitutive relationship.

The use of this type of analysis requires the relations between plastic strain and applied stress for each of the pavement components; i.e.,

$$\epsilon^p = f(\sigma_{ij}) \tag{13}$$

where

ϵ^p = plastic or permanent strain and
σ_{ij} = stress state.

It is then possible to estimate the permanent deformation occurring in that layer by computing the permanent strain at a sufficient number of points within the layer to reasonably define the strain variation with depth. The permanent deformation is the sum of the products of the average permanent strains and the corresponding differences in depths between the locations at which the strains were determined (Figure 16), i.e.;

$$\delta_i^p(x,y) = \sum_{i=1}^{n} (\epsilon_i^p \Delta z_i) \tag{14}$$

where

$\epsilon_i^p(x, y)$ = rut depth in the i th position at point (x, y) in the horizontal plane,
ϵ_i^p = average permanent strain at depth $[z_i + (\Delta z_i/2)]$, and
Δz_i = difference in depth.

The total rut depth can be estimated by summing the contributions from each layer.

From the knowledge of the plastic strain at various numbers of load repetitions, the development of rutting with traffic can thus be estimated.

This approach has been used to predict permanent deformation in either a portion of or the total pavement structure by Morris and others (51), Barksdale (53), McLean and Monismith (55), Freeme and Monismith (56), Snaith (57), Brown and Snaith (58), Hills and others (59), Chomton and Valayer (60), and Van de Loo (61).

The results of one analysis are shown in Figure 17. Comparisons of this type indicate that the use of elastic theory for the distribution of stress and strain together with a constitutive relationship determined from laboratory repeated-load tests can be used for estimating the accumulation of permanent deformation in asphalt pavements.

The use of creep tests on AC together with elastic layer theory to represent the response of the pavement structure to load is an alternative approach proposed by Hills and others (59) and Chomton and Valayer (60) for the estimation of the amount of rutting occurring in the asphalt-bound layer(s) of a pavement structure.

Observations of the development of rut depth with load repetitions in laboratory test tracks (two-layer pavements consisting of asphalt concrete resting directly on subgrade) provide data that, when suitably transformed, have the same shape as the test results for laboratory creep tests in uniaxial compression [Figure 18 (59)]. The quantities in this figure are estimated as follows:

$$S_{mix}(\text{laboratory creep}) = \sigma/\epsilon_{mix} \tag{15}$$

where

σ = applied creep stress at temperature T = constant,
ϵ_{mix} = axial strain at particular time t, and
S_{mix} = corresponding mix stiffness at temperature T and time t.

$$S_{bit}(\text{laboratory creep}) = \sigma/\epsilon_{bit} \tag{16}$$

where S_{bit} = asphalt stiffness (estimated using the Shell procedure).

$$S_{mix}(\text{rutting test on field pavement}) = z\sigma_0/(Br/H_0) \tag{17}$$

where

Z = f[radius of loaded area/thickness of asphalt-bound layer (H_0), $E_{subgrade}/E_{asphalt\ concrete}$],

σ_0 = tire contact pressure,
r = total rut depth at pavement surface, and
B = proportion of total rut depth in asphalt-bound layer.

For the S_{bit} (rutting test), only the viscous component, $(S_{bit})_v$, is estimated.

$$1/(S_{bit})_v = t/3\eta \tag{18}$$

and

$$3\eta = \lim_{t \to \infty} (t \cdot S_{bit}) \tag{19}$$

For the time of loading in the rutting test,

$$t = nt_w \tag{20}$$

where

n = number of wheel passes and
t_w = time of loading for one-wheel passage.

For different temperature conditions, this becomes

$$(S_{bit})_v = 3/t_w \Sigma(n/\eta)_T \tag{21}$$

The use of this methodology requires the measurement of S_{mix} in the laboratory and the estimation of S_{bit} by the Shell procedure and of $(S_{bit})_v$ from a knowledge of the traffic and temperature conditions and the nomographic procedure to give the rut depth (r) from $Z\sigma_0/(Br/H)$ for the specific pavement conditions. Comparisons between estimated and computed values are shown in Figure 19. While the estimation procedure appears to be applicable to all materials, the computational one has been used only for asphalt-bound materials. However, this should not be considered a limitation, since, as suggested by van de Loo (61), the creep test may become a useful mix-design test to differentiate between different mixtures.

Cumulative Loading Conditions

In the laboratory, it is convenient to apply stresses of a single magnitude to the specific material under investigation.

Because the actual stress sequence in the field is not known, it is desirable to be able to predict the results of cumulative loading from the results of simple loading tests. Monismith and others (62) have suggested such a procedure.

At present, at least two methods are available for this: a time-hardening procedure and a strain-hardening procedure; both are illustrated schematically in Figure 20.

In the time-hardening method, if the specimen is loaded for N_1 repetitions of stress state $\bar{\sigma}_1$, the resulting permanent strain will be $\epsilon_1^p(n)$. The equivalent number of repetitions (N_2') at stress $\bar{\sigma}_2$ that would have given the same permanent strain is obtained as shown in Figure 20, and if a further N_2 applications of $\bar{\sigma}_2$ are applied, the total strain will continue to follow the path.

The strain-hardening procedure requires the determination of ϵ_1^p after N_1 repetitions of stress $\bar{\sigma}_1$. The number of repetitions at stress $\bar{\sigma}_2$ is then taken equal to N_1', and further N_2 repetitions are applied. The total permanent strain is the sum of ϵ_1^p and ϵ_2^p.

Both approaches have been used to predict measured responses in cumulative loading from test data at single stress levels and compared with the experimental results by Monismith and others (62). In general, neither method gives a solution that agrees quantitatively with the experimental results. However, the predicted re-

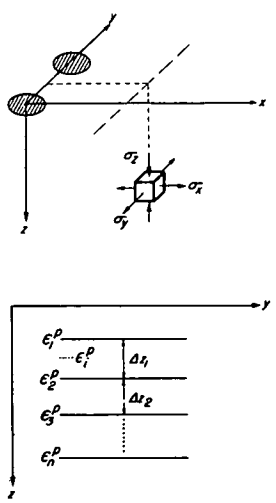


Figure 16. Schematic representation of pavement system used to estimate permanent deformation.

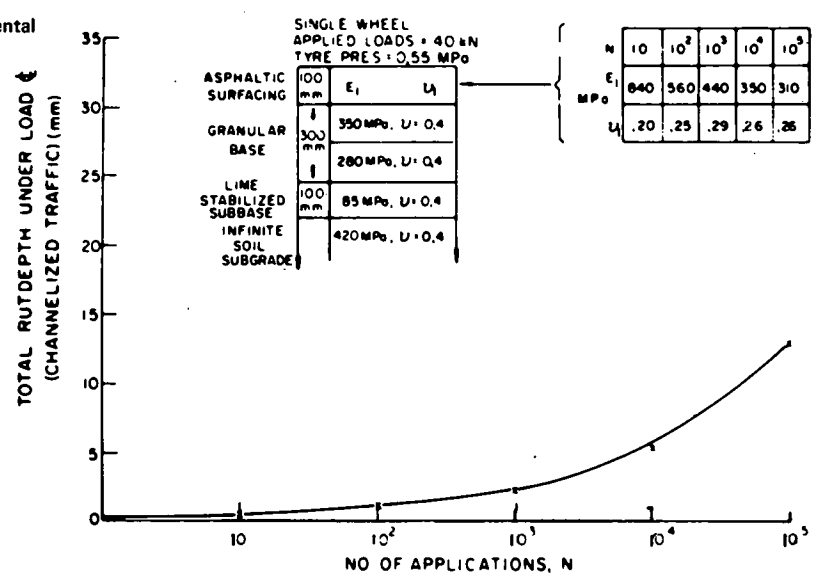


Figure 17. Estimated rutting in experimental test road.

sults are in qualitative agreement and bracket the actual data. The time-hardening procedure gives better agreement if the stress levels are successively increased, whereas the strain-hardening method gives closer agreement if the loads are successively decreased.

Non-Traffic-Load Associated

There are a number of causes of non-traffic-load-associated distortion including (a) volume changes in clays due to changes in suction (or water content); (b) frost heave, which is discussed in a paper in this Special Report by Johnson; and (c) consolidation of soft, compressible underlying layers, but only that due to changes in suction will be discussed here. The resulting heave (or settlement) can be estimated (10) from the profile determined at the time of construction, the equilibrium suction profile, and a relation between water content (or void ratio) and suction for the subgrade soil. If an expansive soil were to be encountered, such an analysis performed in advance of construction could be used to guide the engineer in the selection of compaction conditions to minimize heave.

McKeen has proposed a methodology whereby such estimates can be made. His procedure, as noted earlier, requires additional research.

Fracture (Other Than Fatigue)

A subsystem similar to that shown in Figure 9 can be formulated to examine fracture other than fatigue; Tables 2 and 3 summarize a number of possible causes.

For asphalt pavements, overstress in the asphalt-bound layer can result from braking (decelerating) traffic or from occasional overloaded axles. For PCC, only the latter will lead to fracture.

Pavement structures are also subjected to environmental influences. Such factors, acting by themselves or in combination with load, can also lead to distress. Potential contributing factors to non-traffic-load-associated cracking include volume changes in the PCC on asphalt-bound layer because of temperature changes (or differentials), moisture changes, and (for asphalt pavements) volume changes in the underlying materials, e.g., resulting from curing of cement-treated materials.

For asphalt-bound materials, the tensile strength is dependent on the time of loading and the temperature (which are related to asphalt stiffness as shown in Figure 21). Mix variables, including void content, aggregate gradation, and asphalt content also affect tensile strength.

Heukelom (63) has suggested the following useful procedure for the estimation of the ultimate tensile strength and strain at break of asphalt mixtures that is based on a knowledge of the stiffness of the asphalt in the mix:

$$\sigma_{mix} = M_T \times \sigma_{asp} \qquad (22)$$

where

σ_{mix} = tensile strength of mix,
σ_{asp} = tensile strength of asphalt in mix, and
M_T = mix factor, i.e., f(asphalt content, type and grading of aggregate, mix density or void content).

The breaking strength of the asphalt is a function of its stiffness, which is related to its penetration at 25°C (77°F) and the temperature corresponding to a penetration value of 800 (19). This relation is illustrated in Figure 21. Salam (64) has shown that it will be necessary to determine the mix factor for each mix because the mix characteristics (in addition to asphalt stiffness) permit the development of a range in mixture strengths. However, in the absence of actual test data, the curves shown in Figure 21 can be used for engineering estimates for specific conditions: The curve labeled type 1 is that for conventional dense-graded mixes with void contents in the 4 to 6 percent range and asphalt contents associated with conventional mix design procedures.

The direct tensile strength of PCC may be of interest at early ages of the pavement (65) because it will affect the spacing of cracks during the initial curing. Generally, however, design is based on flexural strength even when the combined effects of traffic-load- and non-traffic-load-associated stresses are being considered.

For cement-stabilized materials, the tensile strength is dependent on type of soil and the curing conditions. Raad and others (66) have shown that values for these materials can reasonably be deduced from the split tensile test. The failure envelope shown in Figure 22 appears to be a reasonable representation for both uniaxial and biaxial stresses.

Fracture Analysis

Fracture: Single Load Application

The fracture potential in an asphalt-bound layer under braking stresses can be analyzed by using multilayer elastic theory (BISAR) and fracture data of the type given here. As with fatigue, stresses can be estimated for specific loading and temperature conditions and compared with data such as that shown in Figure 21.

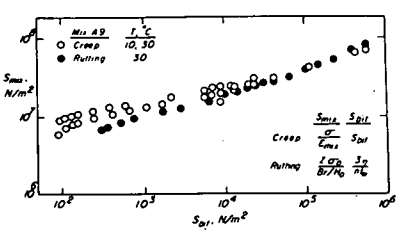


Figure 18. Creep and rutting tests on A9 test track.

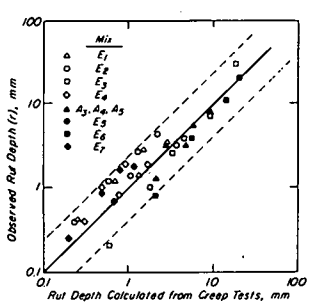


Figure 19. Rut depth observed on tracking machine versus rut depth calculated from creep tests.

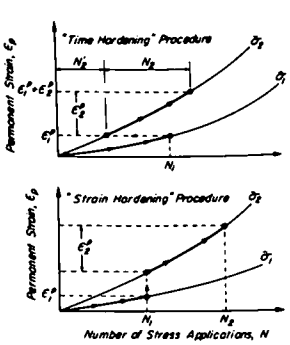


Figure 20. Procedures for the prediction of cumulative loading from simple tests.

Non-Traffic-Load-Associated Fracture

The fracture potential in both AC and PCC due to temperature changes can be estimated.

For PCC, plate theory (67) or the finite element method can be used to estimate these stresses.

For asphalt-bound layers, Christison and others (31) have developed a simplified, yet realistic, method for the estimation of tensile stresses due to temperature changes at low temperatures. This procedure—pseudo-elastic beam analysis—estimates the stresses in the upper part of the asphalt layer where cracking will first develop by using the expression

$$\sigma_x(t) = \int_{t_0}^{t} S(\Delta t, T)\alpha(T)dT(t) \qquad (23)$$

where

$\sigma_x(t)$ = thermal stress,
$S(\Delta t, T)$ = time- and temperature-dependent stiffness modulus, and
$\alpha(T)$ = coefficient of thermal expansion.

In the analysis, the time increment was set at 2 h. The results of a set of stress computations are shown in Figure 23 for a depth of 13 mm (0.5 in) below the pavement surface. Figure 23 also shows the average tensile strength as measured in the split tension test for the corresponding temperature. Cracking is assumed to occur when the computed stress at the 13-mm depth exceeds the fracture strength of the material. Other factors that may contribute to fracture in asphalt pavements include (a) volume changes in the asphalt mixture due to temperature changes and absorptive aggregates and (b) volume changes in the underlying materials due to moisture changes or the curing of cement and lime-stabilized materials. Pretorius (7) has developed a procedure that includes the effects of shrinkage in a cement-stabilized material, but this cannot be implemented as readily as the procedure for the determination of thermal stress.

Figure 21. Tensile strength of asphalt mixes as function of asphalt stiffness, air-void content, and type of mix.

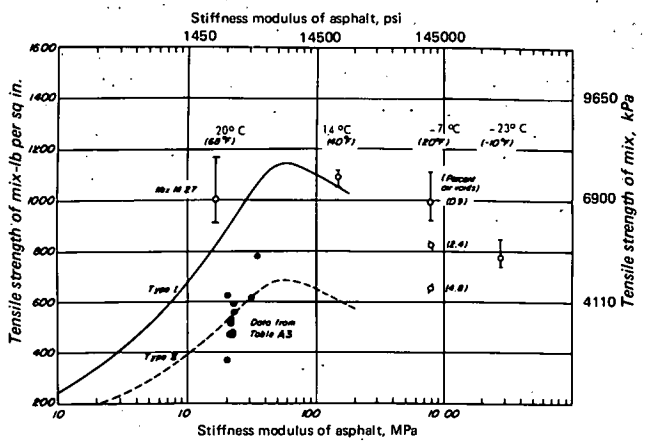
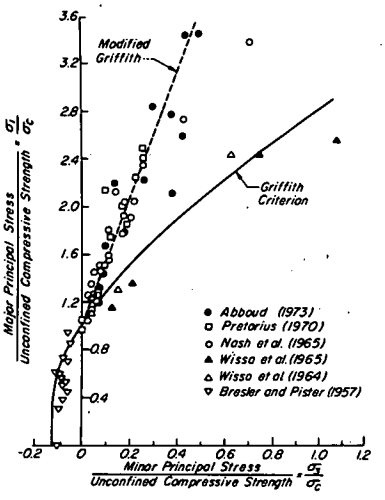


Figure 22. Failure envelope for cement-treated soils.



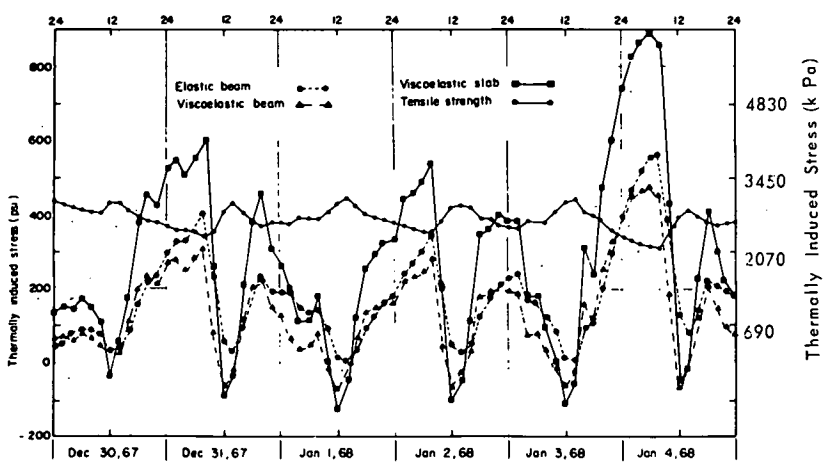
STRUCTURAL DESIGN

In this section, design procedures that have been used for a number of years will be briefly described; the latest methodology associated with each will serve as the basis for the discussion. The recent use of layered elastic theory to develop improved design procedures will also be discussed.

Conventional Design Procedures

There are a number of existing design procedures for airfield pavements [e.g., those discussed by Yoder and Witczak (68) and by Horonjeff (69)], but only a few representative ones will be considered here. These include the CE procedures for asphalt surfaced and PCC pavements, which have been adapted for use by the FAA,

Figure 23. Thermally induced stress at 13-mm (0.5-in) depth of LV 150-200 AC pavement surface of structure D.



and the PCA procedure for PCC pavements.

Asphalt Concrete (Flexible) Pavements

The CBR method of design (25), developed originally by the California Division of Highways, was adopted for military airport use by CE shortly after the outbreak of World War II (70). The method has been modified over the years and is currently used for the design of civilian as well as military airports and for the design of pavements subjected to highway loading conditions. The essential elements of the most recent procedure are summarized below.

The principal design consideration is that the pavement thickness required above a specific layer is related directly to the strength of that layer (as measured by its CBR) and to the applied loading (which includes the effects of magnitude of load, tire pressure, number and spacing of tires, and load repetitions). This is expressed mathematically by Equation 8

$$T = \alpha_i[(ESWL/8.1\ CBR) - (A/\pi)]^{1/2} \quad (8)$$

in which, for airfield pavements, CBR ≤ 15.

The materials property needed for this method is the CBR. It is measured for a series of water contents and dry densities for both the soaked and the unsoaked condition. The compactive effort used in preparing the test specimens ranges from that of the standard to that of the modified American Association of State Highway and Transportation Officials tests.

Portland Cement Concrete (Rigid) Pavements

Both CE (27) and PCA (14) use the Westergaard analysis as the basis for their design procedures. The CE uses an edge-loading condition and, in the PCA procedure, the load is assumed to be applied to the interior of the slab.

In the Westergaard analysis, the modulus of the subgrade reaction can be determined from plate load tests (CE and PCA use different criteria to define k) or estimated from other test parameters (14). In the CE procedure, fatigue in concrete is considered through a design factor (44) that is applied in the same manner as the safety factor in the PCA procedure (14), with both procedures selecting a pavement thickness for a particular design aircraft. Figure 12 illustrates a comparison of the fatigue relations in both design procedures.

The PCA procedure can also incorporate the effects of mixed traffic. Stresses associated with different aircraft can be estimated and their effects combined by the linear summation of cycle ratios.

The essential elements of both procedures are summarized below.

The principal design considerations of the CE procedure are that

1. The pavement thickness is a function of the modulus of rupture of the PCC, the applied load, and the stiffness of the underlying subgrade (as measured by the modulus of subgrade reaction, i.e., k);
2. The Westergaard analysis (a plate on a dense liquid subgrade) for an edge-loading condition is used, and a 25 percent load transfer to the adjacent slab is assumed; and
3. To allow for fatigue, a design factor is introduced that is defined as concrete flexural strength/edge stress and is dependent on the number of coverages.

The materials properties needed for this method are k, which is determined from plate-loading tests, and the 90-d modulus of rupture of the concrete, which is determined from a third-part flexural test.

The principal design considerations of the PCA procedure are that

1. The pavement thickness is a function of the modulus of rupture of the PCC, the applied load, and the stiffness of the underlying subgrade (as measured by k);
2. The Westergaard analysis for an interior loading condition, which is available in computerized form, is used; and
3. The effects of fatigue are considered by the use of either a safety factor (e.g., 1.7 to 2.0 for critical areas) or a linear summation of cycle ratios (when mixed traffic loading can be applied).

The materials properties needed for this method are

1. k, which is determined from plate-loading tests;
2. Ninety-d modulus of rupture of concrete, which is determined from a third-part flexural test; and
3. General fatigue curve, in which the ratio of the applied stress to the modulus of rupture is related to the number of stress applications.

Some modifications, as discussed by Parker in a paper in this Special Report, are required for CRC and fibrous concrete pavements. When CRC pavements are used as overlays, elastic layer theory is used to estimate stresses. Parker also discusses a tentative design procedure that is available for prestressed concrete pavements, based on a procedure originally presented in 1961 (71), but the joint problems have not been completely resolved.

Design Procedures Based on Elastic Layer Theory

Some of the subsystems discussed above have been combined into design procedures for both highway and airport pavements. Three such procedures developed by Shell, AI, and WES for FAA are discussed below.

Shell Procedure

This method of pavement design was originally developed for highway pavements and later adapted to airport pavements (15). It is applicable to pavements having AC resting on granular materials that in turn rest on subgrade soils whose stiffness (modulus) can be defined approximately by an estimated or measured CBR value. The procedure is also applicable to the selection of the thickness of asphalt pavements resting directly on subgrade. The procedure is summarized below [1 MPa = 145 lbf/in^2 and °C = (°F - 32)/1.8].

The principal design considerations are that

1. The pavement structure is represented by a three-layer elastic system (full friction at layer interface) for which the critical design conditions are (a) the horizontal tensile strain (ϵ_t) on the underside of the asphalt-bound layer (if this is excessive, cracking may occur in the asphalt layer) and (b) the vertical compressive strain on the surface of the subgrade (if this is excessive, permanent deformation at the pavement surface will occur) and
2. For design aircraft at 1 000 000 applications, ϵ_t = 0.023 percent (0.000 23 in/in) and ϵ_v = 0.09 percent (0.000 9 in/in); design charts have been developed for individual aircraft (i.e., mixed traffic is not considered).

The materials in each of the three layers are assumed

to be homogeneous, isotropic, and elastic. Their properties needed for this method are

1. For the AC—(a) time-of-loading and temperature dependence; (b) tensile strains, which are determined for an assumed stiffness of 6.2 GPa (900 000 lbf/in^2) that corresponds to a temperature of 10°C (50°F) and a time of loading of 0.02; and (c) subgrade strain, which is determined by assuming an air temperature of 35°C (95°F) and an effective stiffness modulus in the range of 1.1 to 1.4 GPa (150 000 to 200 000 lbf/in^2), depending on the asphalt layer thickness selected;
2. For the untreated aggregate base—modulus of the granular base, which is dependent on the subgrade modulus and the thickness of the base layer; and
3. For the subgrade soil—an approximate relation between modulus and CBR that has been established by in situ vibratory testing [e.g., $E_{subgrade}$ = 10 CBR (MPa) [1500 CBR (lbf/in^2)]]. Figure 24 (15) illustrates a typical design chart associated with this procedure.

Asphalt Institute Procedure

This procedure (16) is limited to AC resting directly on a prepared subgrade (two-layer system). It is illustrated in Figure 25. The design considerations are that

1. The pavement structure is represented as a two-

Figure 24. Design curves for B747.

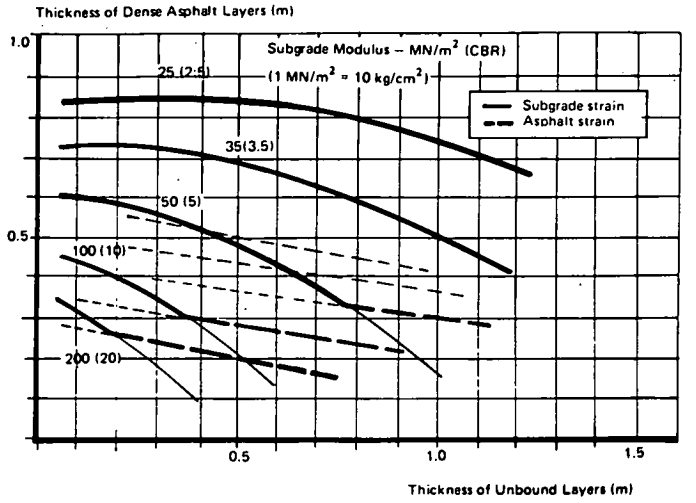
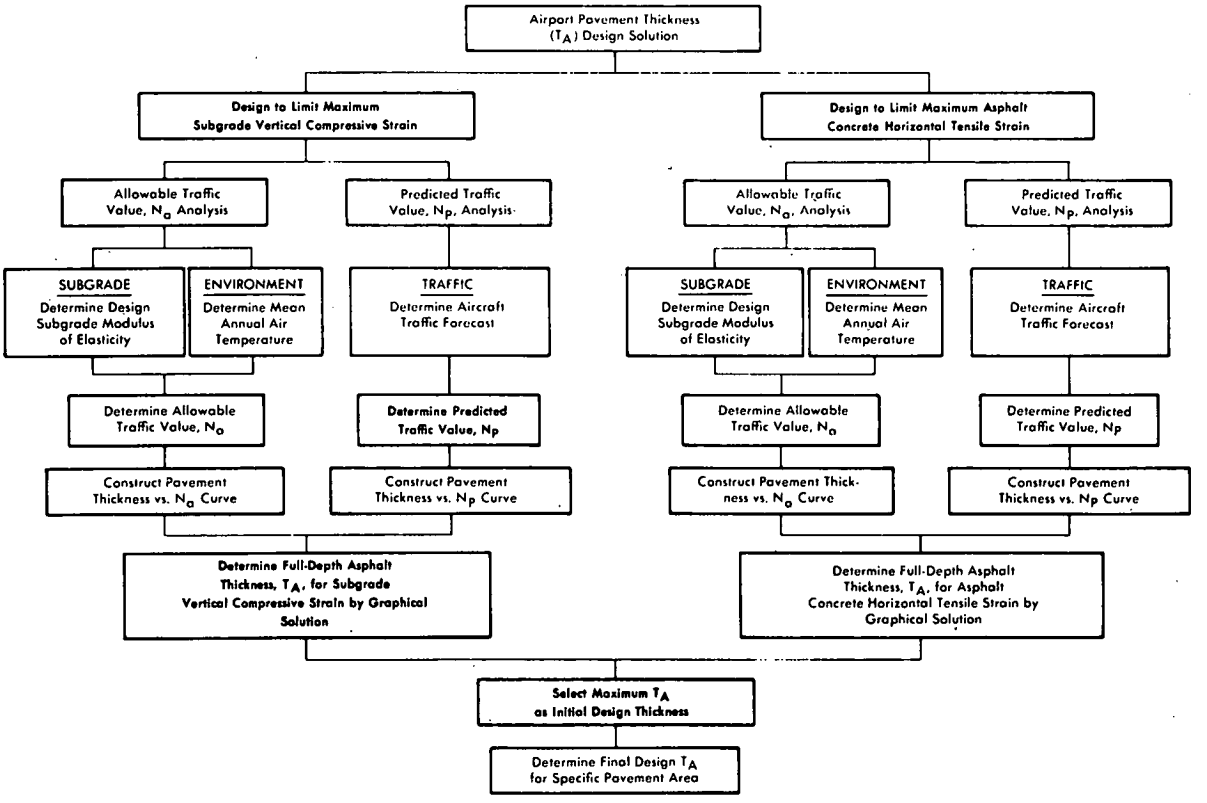


Figure 25. Flow chart for design of airport pavement thickness.



layer system consisting of AC resting directly on a prepared subgrade, for which the critical design conditions are that (a) the horizontal tensile strain on the underside of the asphalt-bound layer is limited to prevent fatigue cracking and (b) the vertical compressive strain at the subgrade surface is limited to minimize the potential for surface rutting, and

2. Traffic is determined in terms of estimated DC-8-63F repetitions.

The materials in each of the layers are assumed to be elastic in response. Their properties needed for this method are

1. Time-of-loading and temperature dependence, which affect AC stiffness;
2. Fatigue characteristics of the AC, which are expressed in terms of strain versus applications to failure for a range in mixture stiffnesses; and
3. Modulus of subgrade, which can be estimated, measured from field tests, or measured from laboratory tests.

This method contains a number of innovative design concepts. For example, it permits an airport pavement to be designed for mixed traffic, the condition representative of most large civilian airports. Environmental effects are included in that the effect of temperature on AC stiffness is recognized.

U.S. Army Engineer Waterways Experiment Station Procedure

This procedure is similar in format to the Shell and AI procedures in that the vertical strain at the subgrade surface and the tensile strain on the underside of the treated layer are the principal design criteria. A flow diagram of one phase is illustrated in Figure 26 (72); it is discussed by Barker in a paper in this Special Report. The principal design considerations are that

1. The pavement structure is represented as a multilayer elastic system with an AC surface and either treated or untreated base and subbase materials,
2. Criteria for limiting subgrade and limiting tensile strain in treated layers are included, and
3. Traffic is expressed in terms of equivalent operations of a design aircraft.

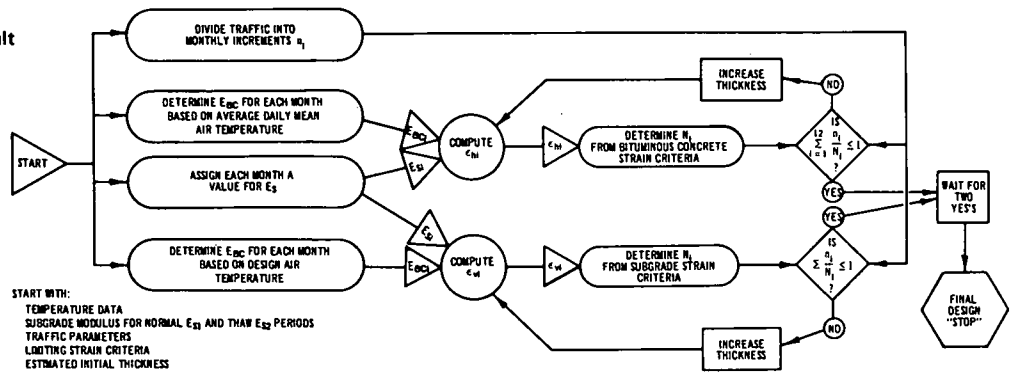
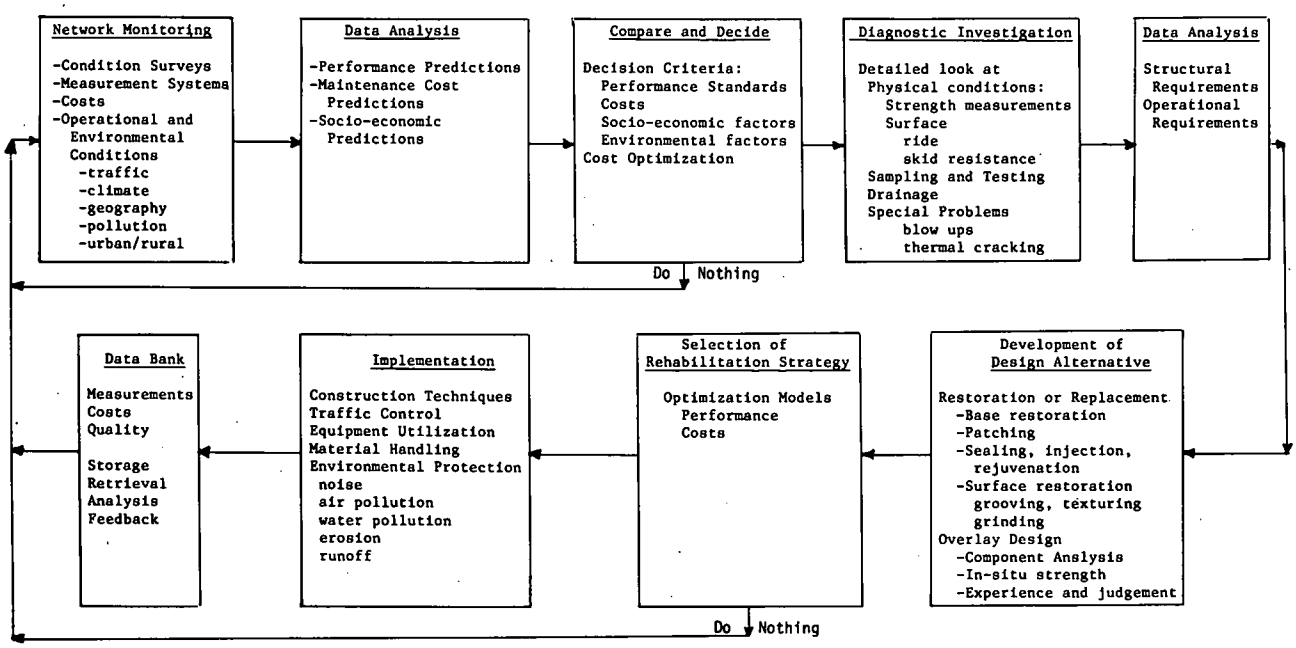


Figure 26. Flow chart of important events for asphalt concrete pavement.

Figure 27. Research framework for pavement rehabilitation.



Note: Sub-items in each of the nine elements are not implied to be complete, but are indicative of the type of factors or conditions involved.

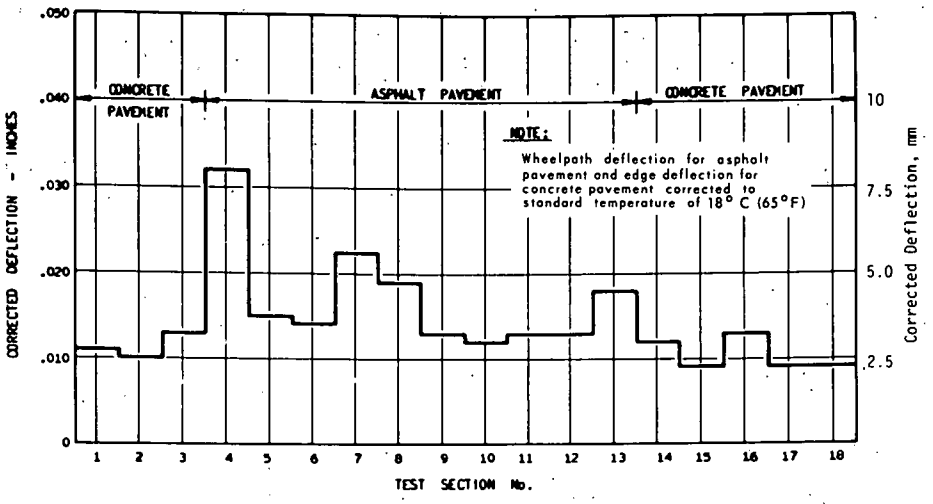
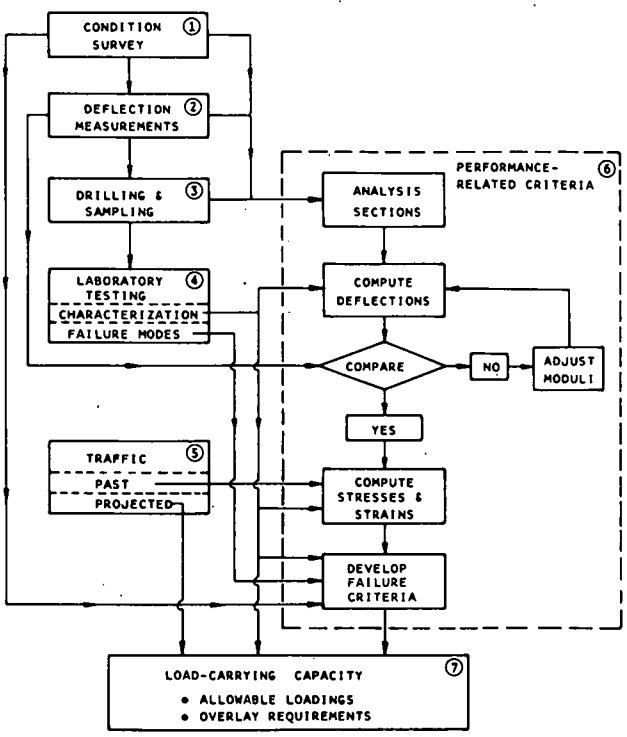


Figure 28. Average Benkelman beam deflections under 40-kN (9000 lbf/in^2) dual wheel load; Salt Lake City instrument runway.

Figure 29. Method for pavement evaluation.



The materials in each of the layers are assumed to be elastic in response. Their properties needed for this method are

1. The effect of temperature on the stiffness characteristics of the AC;
2. The subgrade modulus, which is determined by a resilient modulus test (changes in subgrade stiffness due, e.g., to frost action can be included); and
3. Modulus characteristics of untreated granular materials, which are defined in terms of the subgrade thickness and layer thickness.

PAVEMENT PERFORMANCE EVALUATION AND STRUCTURAL REHABILITATION

The rehabilitation of airport pavements is assuming increased significance and requiring an increased proportion of the funds allocated to these facilities. It is important then to consider the problems involved. Figure 27 (73) provides a concise framework within which rehabilitation can be viewed.

Performance Evaluation

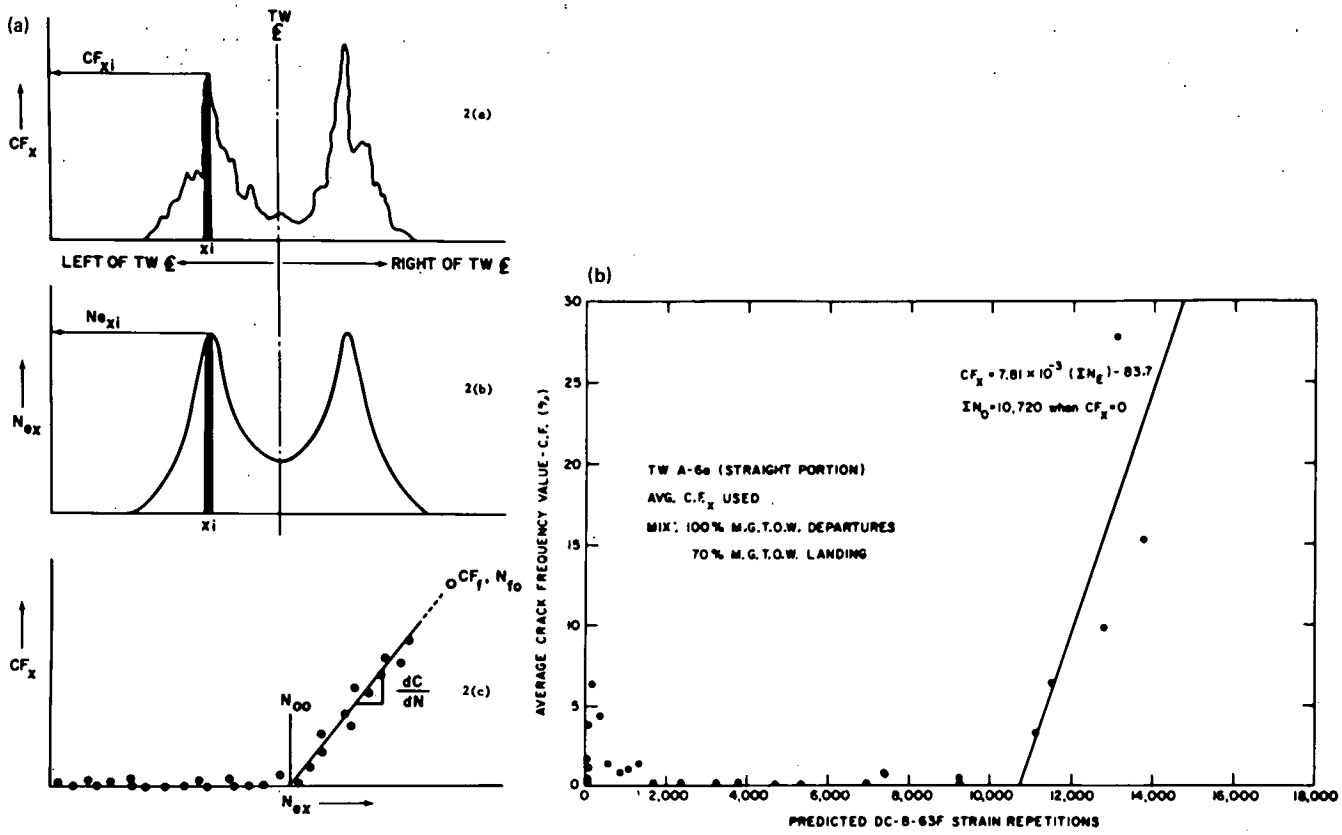
It is important, as noted by Witczak, to consider both the functional and the structural performance of a pavement system. Functional performance is related to how well the pavement serves the user—i.e., the aircraft and its occupants. If the pavement becomes too rough, it will be difficult to operate the aircraft, and the skid and hydroplaning characteristics may be affected. While the functional and structural characteristics are related, no well-defined relation between structural distress (of the type discussed above) and functional performance has yet been established. It is necessary to measure both the functional performance and the structural performance on a systematic and continuing basis and use judgment to decide when structural deterioration will lead to a level of functional performance below that considered reasonable.

Techniques are now available to measure the longitudinal profile and to relate this to the vertical accelerations in the aircraft as it traverses the runway. In this Special Report, this is discussed by Gerardi, and Sonnenburg presents an alternative procedure. Skid resistance can also be physically measured, and hydroplaning potential can be assessed primarily on the basis of visual examination. When (a) the pavement becomes excessively rough, (b) its skid resistance decreases to below some tolerable level, or (c) the potential for hydroplaning increases, some form of rehabilitation must be accomplished—e.g., grooving of the existing surface or the application of a porous friction course or an overlay.

The major reason for measuring the structural performance of a pavement is to attempt to anticipate when rehabilitation should be accomplished so that its functional performance will be maintained at a reasonable level. This is a difficult problem requiring considerable judgment. Fortunately, however, many of the concepts discussed above can be used to assist the engineer in making the necessary decisions if the proper structural measurements have been obtained.

Until recently, the structural evaluation of airport pavements required time-consuming destructive test procedures (8). However, nondestructive procedures are now being developed, although sole reliance should not

Figure 30. Development of distress criteria: (a) general method for determining taxiway-distress pattern and (b) relation between crack frequency and repetitions.



be placed on such procedures. Small test specimens of representative layers can be quickly obtained and used as described above.

The equipment generally used for the determination of structural response measures the surface deflection due to either slow moving or vibrating loads. There is also equipment that measures the propagation of waves from vibrating sources applied to the pavement surface. The deflection-measuring devices include (a) Benkelman beam, (b) traveling deflectometer (State of California), (c) LaCroix deflectograph (Laboratoire des Ponts et Chausées (Paris), France; U.K. Transport and Road Research Laboratory, Great Britain; and National Institute for Road Research, South Africa), and (d) dynamic deflection devices such as the Dynaflect, Road Rater, and the CE vibratory equipment.

The results of measurements with these devices can be used both to categorize pavements into sections of comparable response [e.g., the measurements made on the Salt Lake City instrument runway shown in Figure 28 (17)] and to deduce pavements that can be used for structural evaluation and overlay design purposes.

Unfortunately, difficulties in the interpretation of the dispersion curves (wave velocity versus wavelength relations) obtained from surface vibratory measurements have so far prevented general implementation of this procedure to deduce structural properties (74).

Pavement Overlay Design

Deflection measurements have been used as part of the overlay design process to categorize airport pavements into representative sections so that more detailed structural information can be obtained for overlay design (17). Recently, dynamic stiffness measurements from vibratory loading have been incorporated by CE in their procedure for the design of overlays for flexible and rigid pavements. In this Special Report, this technique is described by Hall, Yang describes an alternative procedure—the frequency sweep method—that uses the dynamic deflection data as a part of the evaluation and overlay design process, and Barenberg attempts to place the procedures described by Hall and Yang in a proper perspective relative to the entire problem of nondestructive evaluation.

Thus, while nondestructive structural evaluation is becoming a part of the airport pavement management framework, I believe that to limit structural evaluation for rehabilitation purposes to the nondestructive procedures currently available is premature. The following discussion illustrates this point.

The results of the research reported above can be effectively used to extend the bounds of overlay design. One such framework is illustrated in Figure 29 (17) and can be briefly summarized as follows:

1. The condition of an existing pavement, including the nature and extent of its distress, is carefully ascertained. The information is stored in a data bank (Figure 27) where it is readily accessible or is shown on a large-scale plan of the airport facilities. This information is used in the sampling and field-test phases and in establishing performance criteria for related distress.
2. Deflection measurements are made to measure the pavement response under known loading conditions so that areas of significantly different structural responses can be categorized.
3. Pavement cores, layer samples, and undisturbed subgrade samples are obtained to varying depths depending on the facility together with information on such fac-

tors as layer thicknesses. (This process need not cause much delay to air traffic because the sampling holes are small.)

4. Laboratory testing is carried out to determine representative stiffnesses (moduli) by a form of dynamic or repeated-load testing and to establish failure (distress) criteria where appropriate.

5. Detailed examination of past traffic, anticipated traffic projections, and assignments to specific taxiways are made. [The equivalency concept (4) can be useful in this analysis.]

6. From the information established in the previous steps, performance-related criteria are established (5, 17). [Figure 30 illustrates an application to the Baltimore-Washington International Airport (5).] For example, by using the laboratory properties measured in step 4 as the initial input, deflections under the known loadings can be estimated, and by comparing these values to those measured in step 2, adjustments in the laboratory-determined stiffness values can be made until the predicted and the measured deflections are in reasonable agreement. The critical performance parameters for the vehicle or vehicles in question are then determined by suitable analysis (e.g., elastic layer theory) and related to acceptable and not acceptable performance areas determined in the condition survey (step 1) and to the laboratory-determined failure criteria (step 4).

7. Either the load-carrying capacity of the existing pavement or the projected overlay requirements for future traffic are thus estimated. If load-associated cracking is the distress mode under consideration, remaining-life estimates in the existing pavement can

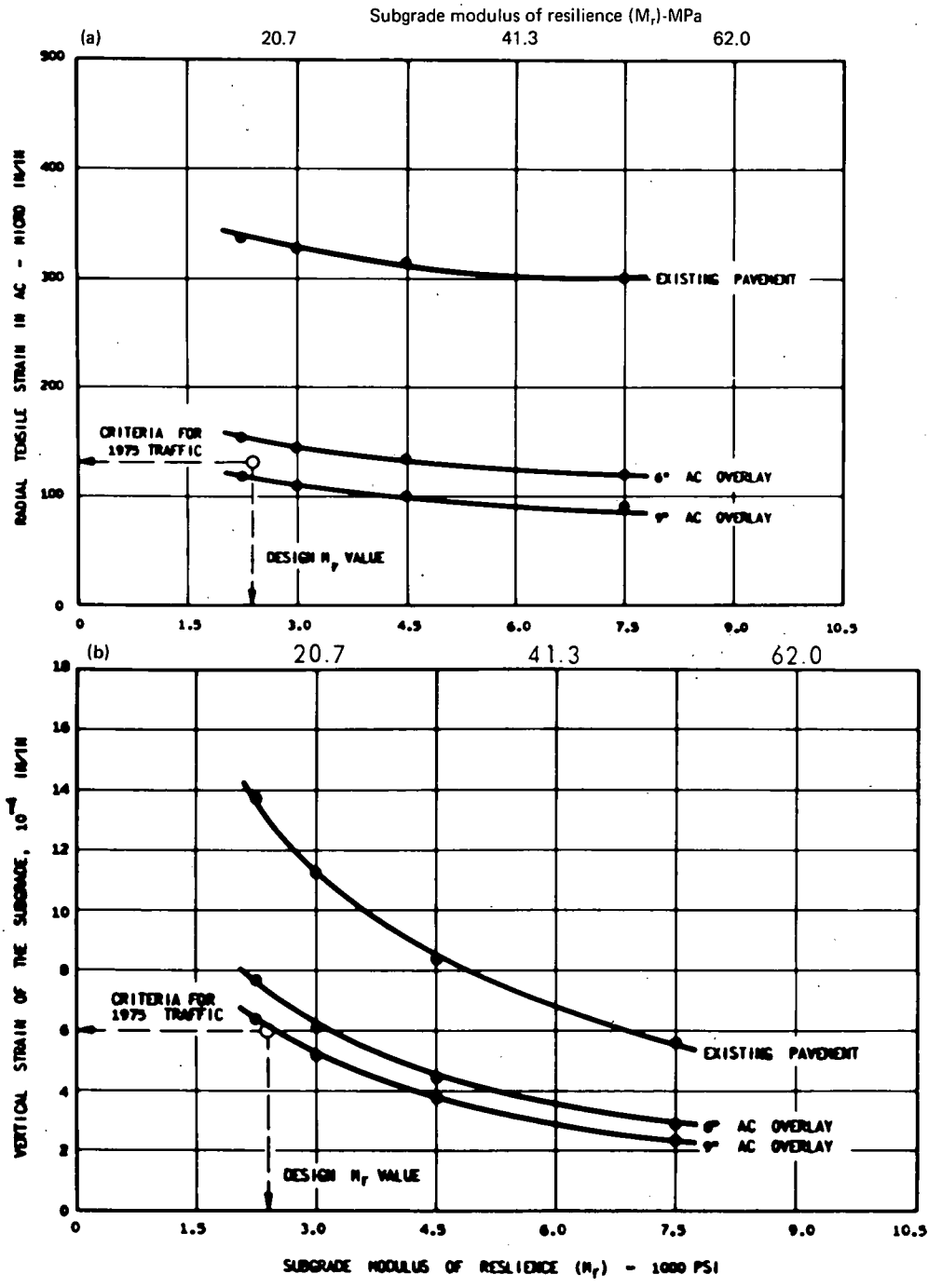


Figure 31. Calculated overlay requirements for Boeing 747: (a) based on strain in asphalt concrete and (b) based on vertical strain on subgrade.

be ascertained by using the concepts described above. [Witczak (5) has successfully applied this concept in his analysis of the Baltimore-Washington International Airport; his recommendations are given in Tables 4 and 5. The results of an analysis for overlay requirements for the Salt Lake City Airport are shown in Figure 31 (17).]

Maintenance Management

Recently, a number of management systems for pavement maintenance have been developed (75, 76) that define and evaluate optimal overlay maintenance strategies for in-service AC pavements.

The framework developed by Smith (76) is illustrated in Figure 32. In this system, a Markov decision model was developed to define optimal overlay maintenance strategies. This model was initially quantified by using the subjective opinions of experienced highway engineers, and provisions were made to update the initial estimates with field data by using Bayesian statistics. Optimal maintenance strategies were determined by minimizing the expected present value of total costs associated with a pavement and were considered to consist of the highway department maintenance costs plus the excess-user costs. In this procedure, alternatives were considered to be thin, medium, and thick overlays. The results indicated that when overlay maintenance was required, medium or thick overlays representative of 10- and 20-year design periods respectively were optimal.

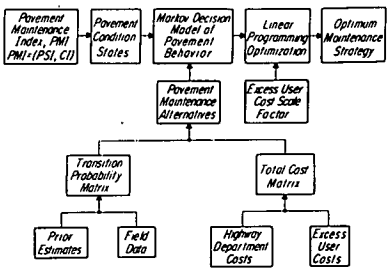


Figure 32. Schematic diagram of pavement-maintenance management system.

Pavement-maintenance management systems of this type provide a systematic and reasonable way to define overlay maintenance policies and develop long-range plans for future maintenance as well as a way to determine the optimal utilization of available funds.

Similar systems could be developed for airport pavements and would probably be much simpler because of the limited area as compared to highway pavements.

CONCLUDING REMARKS

I hope this introduction to airport pavement management will place the materials presented at the conference in a proper perspective. Much information has been developed in recent years that permits an increase in the scope of pavement design and rehabilitation and will (a)

Table 4. Overlay requirements for Baltimore-Washington International Airport.

Taxiway Section	Overlay Thickness Required Now to Provide Service to Future Year (mm)				
	1976	1981	1986	1991	1996
A-1	–	–	15.2	30.5	53.3
A-2	12.7	33.0	66.0	96.5	132.1
A-3	55.9	101.6	137.2	172.7	203.2
A-4	53.3	91.4	124.5	157.4	190.5
A-5	38.1	73.7	106.7	137.2	172.7
A-6a	76.2	114.3	150.0	182.9	210.8
A-6b	109.2	157.5	195.6	231.1	271.8
B-2	–	12.7	25.4	45.7	78.7
C-1	50.8	91.4	127.0	157.5	200.7
C-2	63.5	104.1	139.7	170.2	213.4
D-1	30.5	68.6	104.1	139.7	172.7
D-2	–	–	10.2	17.8	25.4
D-3	20.3	40.6	68.6	99.1	132.1
D-4	20.3	40.6	68.6	99.1	132.1
E-1	–	–	15.2	30.5	53.3
E-2	–	15.2	30.5	53.3	83.8
E-3	35.6	68.6	101.6	134.6	165.1
E-4	81.3	124.5	160.0	193.0	226.1
F-1	111.8	160.0	198.1	236.2	274.3
F-2	81.3	124.5	154.9	195.6	233.7
F-3	76.2	121.9	154.9	193.0	231.1
G-1	–	10.2	22.9	40.6	66.0
H-1	–	–	–	–	–

Note: 1 mm = 0.039 in.

Table 5. Recommended overlay design program.

Overlay Priority	Description	Recommended Overlay Thickness (mm) Year to Provide					
		1975	1975-1978	1978-1980	1980-1983	1983-1985	1985-1988
Runway							
1	RW 10-28	127	–	–	–	101	–
1	RW 15-33	101	–	–	–	76	–
2	RW 4-22	–	–	–	76	–	–
Taxiway							
1	TW F-1	127	76	–	–	–	76
1	TW A-6b	127	76	–	–	–	76
2	TW F-2	127	–	–	101	–	–
2	TW F-3	127	–	–	101	–	–
2	TW E-4	127	–	–	101	–	–
3	TW C-2	101	–	101	–	–	–
3	TW A-6a	101	–	101	–	–	–
3	TW A-3	101	–	101	–	–	–
4	TW C-1	101	–	–	–	101	–
4	TW A-4	101	–	–	–	101	–
5	TW A-5	–	101	–	–	76	–
5	TW D-1	–	101	–	–	76	–
5	TW E-3	–	101	–	–	76	–
6	TW A-2	–	–	76	–	–	76
6	TW D-3	–	–	76	–	–	76
6	TW D-4	–	–	76	–	–	76
7	TW E-2	–	–	–	101	–	–
8	TW B-2	–	–	–	76	–	–
8	TW G-1	–	–	–	76	–	–
9	TW A-1	–	–	–	–	76	–
9	TW E-1	–	–	–	–	76	–
9	TW D-2	–	–	–	–	76	–
10	TW H-1	–	–	–	–	–	–

Note: 1 mm = 0.039 in.

Table 6. Summary of conference materials.

System Phase	Category	Specific Item	Presenter	FAA Report and Date
Input	Traffic	Lateral distribution on runway	V. HoSang	FAA-RD-74-36, Feb. 1975
		Dynamic load effects	R. Ledbetter	FAA-RD-74-39-III, June 1976
		Aircraft gear loads	R. C. O'Massey	—
	Environment	Frost	T. Johnson	FAA-RD-74-30, Oct. 1974
		Volume change	R. G. McKeen	FAA-RD-76-66, Jan. 1976
	Materials	Portland cement concrete: (a) continuously reinforced concrete, (b) fibrous concrete, and (c) prestressed concrete	F. Parker	FAA-RD-74-37; FAA-RD-74-31, Nov. 1974; and FAA-RD-74-34-II, Nov. 1974
		Porous friction course	T. White	FAA-RD-73-197, Feb. 1975
			L. Duggan	—
		Soil classification	F. Horn	FAA-RD-73-169, May 1974
		Materials variability	R. H. Brown	—
			E. H. Brown	FAA-RD-73-199, Oct. 1975
		Load transmission capabilities (layer equivalencies)	G. M. Hammitt	FAA-RD-73-198-II
	Construction	Quality control	E. H. Brown	FAA-RD-73-199, Oct. 1975
			R. H. Brown	—
			T. R. Wathen	—
		Materials consideration		
		Fibrous concrete	R. Lowe	—
		Porous friction course	T. White	FAA-RD-73-197, Feb. 1975
			F. Parker	FAA-RD-74-37; FAA-RD-74-31, Nov. 1974; and FAA-RD-74-34-II, Nov. 1974
Pavement structural model and mechanical state	Thickness selection	Portland cement concrete	F. Parker	FAA-RD-74-37; FAA-RD-74-31, Nov. 1974; and FAA-RD-74-34-II, Nov. 1974
		Improved methodology	W. Barker	FAA-RD-74-199, Sept. 1975
		Design for frost	T. Johnson	FAA-RD-74-30, Oct. 1974
		Design for swelling soils	R. G. McKeen	FAA-RD-76-66, Jan. 1976
Decision criteria	Pavement and aircraft costs	Airport pavement compatibility study	R. Hutchinson	FAA-RD-73-206, Sept. 1974
Performance	General considerations	Framework	M. Witczak	—
	Functional evaluation	Aircraft response to pavement unevenness	A. Gerardi	FAA-RD-76-64
		Unevenness measurement and criteria	P. N. Sonnenburg	FAA-RD-74-188, April 1975; and FAA-RD-75-110
	Structural evaluation	Nondestructive evaluation	V. W. Hall	FAA-RD-73-205-I, Sept. 1975
			N. Yang	—
			E. Barenberg	—
	Example	O'Hare Airport	D. M. Arntzen	—

ensure effective use of marginal or new materials, (b) ascertain the effects of changing mixes in aircraft or of increased operations of specific aircraft, and (c) more effectively engineer large or unusual projects.

Table 6 summarizes the materials presented at the conference in a framework such as that shown in Figure 1.

Some concern might be expressed regarding the time and cost associated with the use of some of these concepts. I believe that we should recognize that pavement design and rehabilitation should not necessarily be expressed in the form of simple charts or tables (I specify "not necessarily" because, under some circumstances, it may be worthwhile and expeditious to do so for reasons of cost and convenience; however, under other circumstances, such simplification is undesirable and the best engineering approach should be used).

Should not the cost of design of an airport runway be comparable to that of other engineering structures of comparable value? I believe that it should; one can envision more effort than the selection of thicknesses or other characteristics with the assistance of a few charts.

It is extremely important to follow through the pavement design process for the construction and maintenance of the resulting facility. If, for example, care is not exercised in the control of the construction process, no matter how good the design, the pavement structure will not perform as expected. Thus, design and construction must be considered together as the design process.

The results of this research also provide a basis for the study of pavement rehabilitation. Fortunately, we have this theory to assist in developing sound rehabilitation practices! Many of the decision-making processes that have been developed in other areas provide us with the methodology with which to develop reasonable maintenance strategies that will effectively use our limited monetary resources.

Much of this methodology is not as well developed as would seem desirable, but let us begin to use it as quickly as possible. For example, in performance and evaluation, we should be using existing technology now on a systematic and continuing basis to develop performance data rather than waiting for the universal procedure. As improved techniques are developed, they can be incorporated into the system.

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Procedures for Airport Pavement Management

Donald M. Arntzen, Bureau of Engineering, Chicago Department of Public Works

The results of runway surveys at Chicago-O'Hare International Airport are summarized, and the causes of the deterioration of these runways are evaluated. Procedures for their rehabilitation and reconstruction are described.

One of the goals of airport pavement management procedures is economy. This encompasses all costs—loss of revenue, delays to aircraft and passengers, aircraft maintenance, pavement maintenance, pavement strengthening, and reconstruction or new construction.