INTRODUCTION TO A RIGID PAVEMENT DESIGN PROCEDURE

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An introduction to a design procedure for rigid airport pavements is presented. The design criteria used in the procedure are based on the tensile stress in the portland cement concrete (PCC) slab as computed by the layered elastic theory and on the strength of the PCC slab as measured by the flexural beam test. The criteria were developed by the analysis of some 60 test sections. In the basic report, procedures are given for the characterization of the pavement materials both by laboratory testing and by typical values and/or correlation studies. The thickness requirements as determined by the new criteria are compared with the thickness as determined by the Corps of Engineers and the Federal Aviation Administration design procedures. The comparison indicated a basic difference between the new criteria and the criteria presently being used. A detailed study of the two criteria shows that the new criteria fit the test data better than the present criteria.

Background

Recognizing that a need for more responsive methodology to handle problems associated with the design of pavements involving composite systems, stabilized layers, and other layered subgrades, and that a broad and technological base was available, the Federal Aviation Administration and the Department of the Army Office, Chief of Engineers, authorized the U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, in 1973 to undertake an investigational program to formulate a "rational" design system for airport pavements.

Development of a structural design system for flexible airport pavements is reported by Reference 1. The procedure for the design of flexible airport pavements is a mechanistic procedure, which utilizes laboratory-determined material properties and the layered elastic theory to compute pavement response parameters that are relatable to pavement performance. A similar approach for the design of rigid pavements will permit consideration of several design aspects that are ignored or approximated in available procedures. Nearly all design procedures in current use model rigid pavements as a two-layer system; i.e., the portland cement concrete (PCC) surface layer is described as a thin elastic plate, and the underlying material is described as a dense liquid foundation. The supporting characteristics of the underlying materials are quantified by a single constant, referred to as the modulus of soil reaction, that is determined with plate load tests conducted on in situ materials.

The characterization of each layer with fundamental material properties obtained from laboratory tests, as opposed to field tests on in situ material, will permit more flexibility for testing the material at a range of field conditions. The use of a layered model and laboratory tests rather than field plate load tests should permit more tests and, therefore, a better representation of the subgrade.

The development of such a procedure for the design of rigid airport pavements has been completed and reported by Reference 2.

Scope

The study was limited to the structural design of the pavement section, i.e., the selection of the required thickness of pavement layers to carry the design traffic under field conditions. The methodology employed was chosen in the interest of developing a practical and implementable design procedure. The use of such methodology did require restrictions on the use of the procedure, most notable of which is the requirement that the procedure be limited to standard jointed PCC pavements.

Approach

The procedure for the mechanistic design of rigid pavements involves three major elements - material characterization, computation of load response parameters, and relating the load response parameters to pavement performance. The first step toward developing the design procedure was the selection of a response model for computation of design parameters. The particular model chosen would dictate the material parameters needed for input and the response parameters available for relating to pavement performance. In the selection

of the response model, major importance was placed on choosing a model that could lead to the development of a practical and implementable design procedure. With the selection of a response model, material characterization procedures were chosen or developed to provide the necessary input for the response model. The material characterization also had to provide data to be used in relating response to performance. Insofar as possible, the methodology for material characterization is to be by laboratory testing but in keeping with the theme of being practical and implementable; alternatives to extensive laboratory testing have been provided. In the development of the performance criteria, available airport test data were analyzed, and a number of response and material parameters considered for use in the criteria. The criteria chosen represent a regression analysis that best describes the performance of the test sections. These criteria are the performance criteria used in the design procedure for the design of airport pavements.

Selection of Response Model

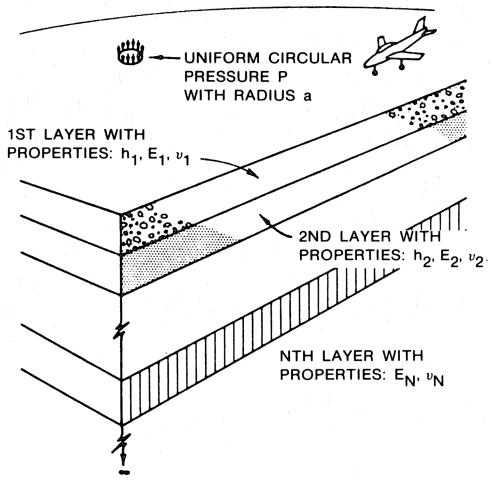
Basis for Selection

The layered elastic model (3) was selected for computing pavement response parameters. By changing from the presently used model (Westergaard idealization), implications are that the presently used model is inadequate and that the layered model offers significant improvements (4,5). These implications may not be totally true and may, in fact, be completely erroneous in certain respects and for certain conditions. The problem stems from the judgment of adequacy or what is best. Such judgment is oftentimes subjective, based on opinion rather than fact, and in many cases, based on a limited range of circumstances. Nevertheless, the selection of the layered elastic response model was based on what was believed to be sound, rational, and practical considerations.

Crawford and Katona (6) provided detailed discussions of the three primary idealizations for modeling rigid pavements and included discussions of various material characterization procedures that are necessary for quantification of properties of the pavement structures. A brief comparison follows in which the primary reasons are outlined for selecting a response model based on the layered elastic idealization.

For the layered elastic idealization (Figure 1 (6, p. 15)), the pavement structure is represented as a series of horizontal, uniform, elastic layers with properties defined by (a) E_i, the modulus of elasticity of the i^{th} layer; (b) ν_i, the Poisson's ratio of the i^{th} layer; and (c) h_i, the thickness of the i^{th} layer. Furthermore, the layers extend horizontally to infinity in all directions, and the n^{th} layer extends vertically to infinity. The Westergaard idealization represents the PCC slab as a thin elastic plate with properties defined by E_p, ν_p, and h_p over a dense liquid (Winkler) foundation. The liquid foundation is characterized as a bed of springs having a certain stiffness. Each individual spring represents the effect of the support provided over a unit area and is quantified by a constant k, which is the ratio of pressure on the unit area divided by the deflection. In the basic Westergaard idealization, loads were represented as uniform circular pressure distributions,

Figure 1. Elastic layered pavement idealization.



but procedures developed by Pickett et al. (7) and Pickett and Ray (8) permit uniform pressure distributions with any shape to be readily handled.

The elastic layered idealization would appear to be a more realistic representation of a real pavement structure since PCC pavements are truly layered systems, although the materials may not be truly elastic. For practical loadings, however, the materials can be represented by quasi-elastic properties. The representation of the top layer as a thin elastic plate (Westergaard idealization) or as an elastic layer is really not that different when the top layer is a PCC slab, as in rigid pavements. The major difference lies in the representation of the remainder of the structure. The use of fundamental constants E and ν to represent the properties of underlying layers is theoretically sounder than a single constant k. From a practical standpoint, it is also more valid, considering that the determination of k is made with a plate test and represents the response of the material to a particular loading condition (i.e., 0.76-m- (30-in.-)diam plate and 68.9-kPa (10 lbf/in.^{2}) vertical pressure), which may be different from that actually experienced in the pavement.

Experience has shown that for single, dual, and even dual-tandem gears with closely spaced wheels on relatively thin slabs (less than about 0.38 m (15 in.)) laid either directly on the subgrade or on granular layers, quantification of the supporting characteristics of the underlying layer with a modulus of soil reaction produces reasonable computations of the response of the pavement. However, for larger loads transmitted to the pavement through a number of widely spaced wheels, for relatively thin, high-strength (large stiffness) base courses, and for thick PCC slabs, the validity of the idealization decreases. For the thicker slabs and widely spaced wheels, the zone of influence (stresses in the underlying material) becomes much larger than that under a 0.76-m-(30-in.-)diam plate, although a 68.9-kPa (10-lbf/in.^{2}) contact pressure may in fact be valid for both conditions. The effect of a thin, high-strength (stiffness) base course will be more pronounced on the load deformation reponse of a 0.76-m- (30-in.-)diam plate than on the load deformation response of a thick PCC slab. The response of the 0.76-m- (30-in.-)diam plate will be significantly reduced by the thin base, whereas the reduction in the response of the pavement will not be as significant.

Another situation in which the use of an elastic layered idealization may be more representative occurs when there exists within the subgrade different types of materials at relatively shallow depths (less than 6 m (20 ft)). For instance, a stiff or a soft layer in the subgrade may not significantly affect the load deformation response of a 0.76-m-(30-in.-)diam plate, but the effect may be significant on the load deformation response of a thick slab loaded with a large load on widely spaced wheels.

Characterization of each layer with elastic constants obtained from laboratory tests, rather than one elastic constant obtained from field tests, provides the designer greater flexibility. At this point, it should be noted that it is recognized that the materials in pavements behave neither elastically nor linearly but that the assumption of linear elasticity is necessary for practical application. The state of stress under which the material is tested in the laboratory may be changed to conform to the most critical state of stress under which it may exist in the pavement. This is contrasted with the constant state of stress at which the modulus of soil reaction is selected. There is also the flexibility of being able to readily change the physical condition of the specimen (moisture, pore pressure, density, etc.), whereas this cannot be so easily accomplished on in situ material. Thus, the use of an indirect correction for saturation of the modulus of soil reaction is not necessary.

The use of laboratory procedures makes it possible to test a more representative sample of the existing subgrade and a larger variety of available base course materials. With the present design procedure, extensive plate bearing tests are the exception rather than the rule. Another factor to be considered is the repetitive nature of the loads applied to a pavement. Certainly, the use of laboratory tests will more readily permit consideration of the effects of repeated load applications than will field plate bearing tests.

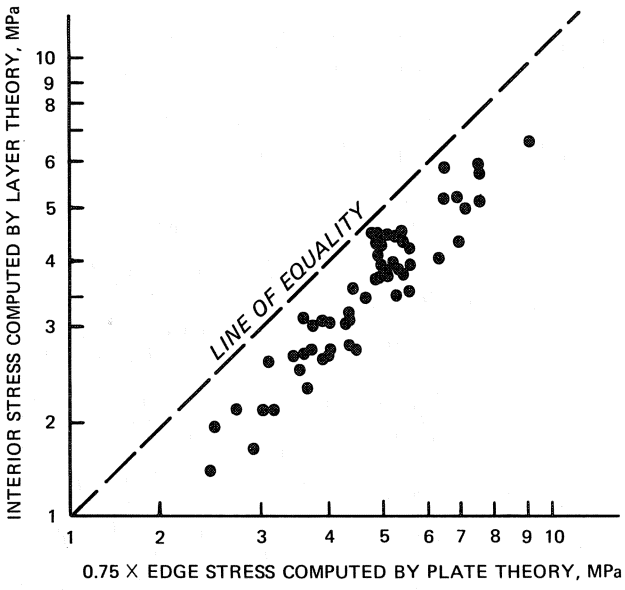
The comparison of the finite element idealization with the Westergaard and elastic layered idealization may not be valid since it refers basically to a computation procedure rather than to a mathematical representation of the physical structure. As has been noted previously, the finite element idealization may be employed for the upper layer with a dense liquid or elastic solid representation for the remainder of the structure. Nevertheless, consideration as a separate idealization has merit for comparison between available techniques for computing the response of a pavement to load. As discussed herein, the finite element representation will mean that the entire structure will be broken into a number of finite elements.

In the finite element idealization, the continuous pavement structure is broken into a number of elements connected at nodal points. The material in each element is assigned properties that may vary from element to element. The number of elements and nodal points that may be considered is limited by computer capacity, and thus boundary conditions must be specified. The loads are applied as concentrated forces at the nodal points. With the aid of special types of elements, discontinuities, special interface conditions, reinforcing steel, and dowel bars may be introduced. Special computational techniques permit consideration of voids and temperature and moisture gradients within the pavement structure. In addition, variable layer properties (thickness and load deformation properties) and nonlinear material response may also be treated.

After consideration of all models, the layered elastic model was selected. The model has several weaknesses, and certainly there are more sophisticated models available. However, it offers a viable alternative that can be implemented into a workable design procedure.

Consider the relationship between the interior stress computed by the layer theory and 0.75 of the edge stress computed by the plate theory (Figure 2). The parameter of 0.75 of the edge stress is the design parameter presently being used for the design of rigid airport pavements. Although the relationship in Figure 2 is not one of equality, it is one of strong correlation; thus, if a workable design procedure could be based on edge stress, then it would be expected that a workable procedure could be developed based on the interior stress.

Figure 2. Comparison of stresses computed from test section data.



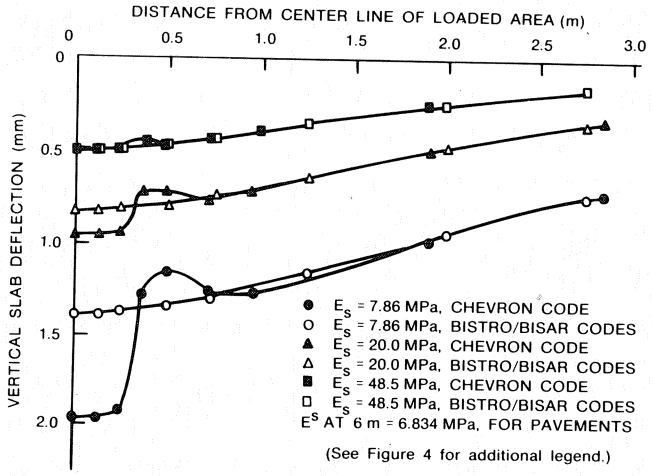
Note: 1 MPa = 145 lbf/in.^{2}.

Selection of Computational Procedure

The two most widely used computer codes to solve for the response of a layered elastic pavement idealization are the BISTRO (9) and the CHEVRON (10) codes. In addition, the CRANLAY (11) and the BISAR (12) codes have been used but not as extensively as the BISTRO or CHEVRON codes. From these, the BISAR code was selected to develop the performance criteria because of the author's confidence in the accuracy of the program and the ability of the program to consider different interface conditions. The BISAR program is an advanced version of the BISTRO program and has capabilities that are not necessary for the use of the design procedure developed in the study but may be useful in the future for pavement analysis. The CHEVRON program does offer some advantages in terms of operation cost and may, in some design situations, be a completely adequate program. As an example, the CHEVRON code has been modified for multiple wheels and is recommended by Barker and Brabston (1) for flexible airport design. In order to maintain as much consistency between pavement types, it was the original intent to use the same code for rigid pavements.

However, for rigid pavements that have a subgrade of a low resilient modulus, the deflection basin computed with the CHEVRON computer code becomes very distorted in the area beneath and immediately adjacent to the loaded area. This distortion is evident when compared with the deflection basins computed using the BISTRO/BISAR computer program (Figure 3). This distortion of the CHEVRON-computed basin is much greater for low subgrade moduli with the distortion decreasing as the subgrade modulus increases until for a subgrade modulus of 48.5 MPa (7000 lbf/in.^{2}) the basins computed are nearly identical. The distortion is caused by inaccuracies in the numerical solution procedures for various integral equations and Bessel functions for large E_p/E_S ratios.

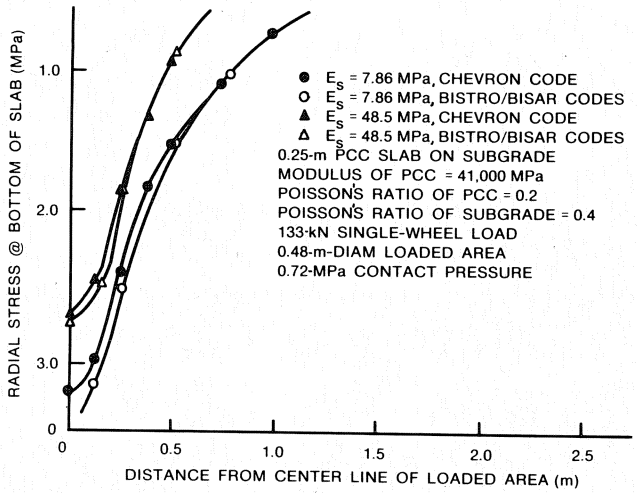
Figure 3. Deflection basins computed with the CHEVRON code compared with the deflection basins computed with the BISTRO/BISAR codes.



Note: 1 mm = 0.039 in.; 1 m = 3.28 ft; 1 MPa = 145 lbf/in.^{2}; 1 kN = 0.225 kip.

A much smaller effect on the stresses is apparent, as illustrated by the plot of radial tensile stress in the bottom of the PCC slab in Figure 4.

Figure 4. Comparison of tensile stress in bottom of PCC slab computed with CHEVRON and BISTRO/BISAR codes.



Note: 1 MPa = 145 lbf/in.^{2}; 1 m = 39.37 in.; 1 kN = 0.225 kip.

However, it was believed that the additional accuracy of the BISTRO/BISAR computer codes was desirable. Because of the cost benefit and additional capabilities of the BISAR code, this program was chosen over the BISTRO code.

Selection of Material Characterization Procedures

Basic Research

The amount of research that has been directed toward characterizing paving materials and subgrades with a test that is more accurate and more fundamental than a plate bearing or CBR (California Bearing Ratio) test is truly prodigious. Conversely, the formulation and application of practical, usable, widely accepted procedures for the routine design of rigid pavements is truly meager. Chou (13) has prepared a state-of-the-art report on the characterization of pavement materials including subgrades. Much of the discussion that follows will be based on this study.

The materials composing a rigid pavement respond neither linearly nor elastically to load, but in a complex manner. Generally, the response is nonlinear and nonelastic for a rather wide range of stress and strain conditions. The approximation of linear elasticity is more valid for such materials as PCC and bound bases (subbases) than it is for granular bases (subbases) and subgrades. The response of many of the materials is highly dependent on the state of stress and the number of load repetitions to which the material has been subjected. Although the response of rigid pavements is not as sensitive to time and temperature (i.e., if the effects of temperature gradients within the PCC slab are ignored) as flexible pavement, temperature and rate of loading will have an influence on the characterization of materials in which a bituminous binder is used or in other layers where freezing occurs.

The pavement designer must then approximate the complex response of materials in order to use a simple analytical procedure to obtain the response of a pavement. As it turns out, many of the necessary approximations are not nearly so drastic as might be suspected from consideration of the total response of the materials. The states of stress and strain within pavements are usually within the range where the assumption of linear response is reasonable. After only a limited number of load repetitions, the assumption of elastic response will usually become more reasonable, although for initial loadings the response may have been very inelastic and nonlinear. The state of stress is variable throughout a pavement structure and will depend on the loading and layer thicknesses. However, a state of stress may be selected which will provide a reasonable approximation of conditions that exist in real airport pavements.

Layers with bituminous binders are beneath the surface, where the variability in the temperature regime is not as great as it is for flexible pavements and the effects of the rate of loading are not as critical. Suitable temperatures and rates of loading can be selected that provide adequate characterization of the material containing a bituminous binder. As to the effect of freezing on the subgrade, the most critical period, in terms of magnitude of pavement response, is during periods of thaw, and these conditions can be approximated in the laboratory.

Procedures will be considered for determining the load deformation characteristics of PCC surface

layers, granular bases (subbases), bound bases (subbases), and subgrades. Details of the procedures for subgrades will depend on whether the material behaves as a cohesive or cohesionless material.

At this time, some discussion of terminology regarding the four types of materials that compose rigid pavements is needed. There is general agreement as to what constitutes PCC surface layers and subgrades. Misunderstandings and the need for more precise definitions arise when considering bound or granular bases (subbases). Chou (13) discusses bound bases (subbases) under the general heading of soil stabilization, i.e., stabilizing agents, stabilization mechanism, purposes for stabilization, and resulting material properties. Wide ranges in each aspect of the stabilization are noted. Barker and Brabston (1) discuss the general area of stabilization and provide limiting values for determining when the stabilizing agents are effective in modifying the properties of the natural material. The one fact that becomes readily apparent from these and other discussions on the subject is that there are many "gray" areas and only a few "black and white." To the myriad of existing definitions and concepts will be added several more, which are peculiar to the procedure contained herein.

Strength and Quality Criteria

Bound bases (subbases) are natural soils, prepared soils (washing, grading, crushing, etc.), or crushed stone, which has portland cement, lime (slaked or hydrated), fly ash, sodium silicate, bitumen, or a combination of the listed ingredients added. A distinction is made between bound bases in which the stabilizing agent is a bituminous binder and one in which another of the listed agents is used. Bituminous stabilized bases depend on the mechanical bond between particles provided by the bitumen binder. The other agents (primarily portland cement, lime, or lime-fly ash) depend on certain chemical reactions to provide the bond between particles and will be referred to as chemically stabilized bases. The distinguishing feature of both types of bases is that the material can be molded into a beam and can sustain flexural-type loadings.

Bituminous bases should meet requirements as set forth in References 14 and 15 for bases in which a bituminous binder is used. The references contain specifications for gradation, amount of binder, etc., which are intended to ensure that the material functions as a bound material.

Chemically stabilized materials should meet requirements set forth in References 14, 16, and 17. Among these are requirements for durability and the requirement that strength increase with age, which are intended to ensure that the materials continue to function with age and that no adverse chemical reactions occur. However, in terms of ensuring that the material functions as a bound material (sustain flexural loading), the requirement that the material attain an unconfined compressive strength of 1724 kPa (250 lbf/in.^{2}) at 28 days, as set forth by Barker and Brabston (1), is applicable for rigid pavements. This requirement should be used in lieu of strength requirements in References 14, 16, and 17.

Those materials that have a chemical stabilizing agent added but do not meet the 1724-kPa (250-lbf/in.^{2}) compressive strength requirement should be characterized with procedures for granular bases (subbases) or subgrades, depending on the nature of the natural material. For instance, a clay subgrade to which lime has been added but which does not meet the 1724-kPa (250-lbf/in.^{2}) strength requirement should be characterized and considered simply as part of the subgrade. The general rule is that the material should be characterized and used in the design as if no stabilizing agent had been added when the compressive strength requirement is not met.

Granular bases (subbases) are natural soils, prepared soils (washing, grading, crushing, etc.), or crushed stone, which meets gradation and durability requirements as set forth in References 14 and 18-21. The characteristic of granular bases (subbases) that distinguishes them from bound bases (subbases) is that they do not possess and/or will not maintain the ability to sustain flexural loading.

The strength of the PCC surface layer will be defined by the flexural strength. This parameter is required in current design procedures, and the same well-established characterization procedures will be used.

Elastic Constants

The elastic constants defined for each layer will be the modulus of elasticity and Poisson's ratio. The modulus of elasticity will receive the greater attention for several practical reasons. One is that, in terms of response of the pavement, the modulus of elasticity is the dominant of the two parameters. A second is that Poisson's ratio varies only within a limited range for the different types of materials composing a pavement structure. Finally, it is difficult to accurately determine Poisson's ratio from laboratory tests.

The modulus of elasticity and Poisson's ratio of PCC are relatively insensitive to state of stress, temperature, or stress repetitions and thus can be represented with a high degree of confidence from the results of laboratory tests. The particular laboratory test considered most appropriate in representing the conditions of the PCC in a pavement system is the flexural beam test, and thus it is the flexural modulus to be used for design. This is not to eliminate the use of other testing procedures for determining a modulus, but it should be stressed that these design criteria are based upon the flexural modulus and that other modulus values should be related to the flexural modulus. There have been numerous studies relating the modulus values obtained from other test procedures, such as the uniaxial compression test, split tensile test, and resonant column test. If the designer has confidence in the correlations to the flexural modulus, then the results of these tests can be used; but the designer should also realize that the level of confidence of the design systems may be somewhat reduced by such correlations. The usage of Poisson's ratio is relatively narrow, and the effects of varying Poisson's ratio over this range on the computed response is almost negligible. Considering the difficulty in measuring Poisson's ratio, a fixed value of 0.2 for PCC will be adequate for design purposes. Thus, this has been the value of Poisson's ratio selected for development of the design criteria.

The modulus and Poisson's ratio values of base material vary over a wide range. A compensating factor is that the principal strength of rigid pavement is derived from the concrete surfacing, and thus relatively large variations in the moduli of the base can be tolerated. For bound bases, since the properties in flexure are considered to be appropriate, the beam test should be employed. Bituminous bound bases in particular are subject to a variation in modulus and Poisson's ratio due to

differences in binder type, binder content, temperature, and rate of loading. There are procedures, as discussed by Barker and Brabston (1) and Chou (13), for estimating the modulus and Poisson's ratio of bituminous bound bases that can be used for design. Such procedures would certainly be appropriate for designs that employ bituminous bases primarily for waterproofing and not as a structural element. Bases stabilized with cement and/or lime will be similar in behavior to PCC; i.e., the modulus will depend on the strength of the material with Poisson's ratio relatively constant. As with PCC, correlations between the properties in flexure and the properties as determined by other laboratory tests have been developed. As with the bituminous bound bases, the modulus of relatively thin bound bases can be crudely estimated without seriously compromising the pavement design. Published values of the modulus of chemically stabilized materials vary over a wide range, i.e., from less than 689 MPa (100,000 lbf/in.^{2}) to that approaching the modulus of lean concrete. For development of these criteria, a modulus value of 1724 MPa (250,000 lbf/in.^{2}) was used. This modulus may be considered typical of what may be expected from a stabilized base. Poisson's ratios for chemically stabilized bases vary over a wider range than the values for PCC, but 0.2 would still be an adequate estimation.

The properties of granular bases are very dependent on the state of stress, state of compaction, moisture content, and to some lesser degree on the aggregate quality. By specifying the material quality and compaction requirements and dealing with moisture conditions through assuming a nearly saturated condition or by applying some rationale for arriving at a design moisture content, the remaining variable influencing the modulus is the state of stress. Since this particular parameter does have such an important effect on the modulus of base materials, a detailed study was conducted on the state of stress under rigid airport pavements. The study indicates that the first stress invariant in bases does not vary greatly and that a value of 68.9 kPa (10 lbf/in.^{2}) would be a typical value applicable for pavement design. The repetitive triaxial test is considered to be the laboratory test that can best be used to establish the characteristics of a particular granular material. Other approximate methods have been developed that do yield reasonable modulus values, but care should be exercised in the use of either the typical state of stress or the approximate methods of determining the modulus. For these procedures to be appropriate, the pavement design should be a typical design and the materials should meet the specified quality and compaction requirements.

Poisson's ratio of granular materials varies with the shear stress. In the range of stress normally encountered in rigid pavement, the value of 0.3 is an appropriate value for design. In the repetitive triaxial test, the Poisson's ratio is particularly difficult to measure and is subject to a high degree of error; thus, in most design situations, no attempt would be made to measure this material property.

Characterization of subgrade soils is normally considered much more crucial than characterization of the bases, primarily because of the relative thickness between the two and because of greater variation, which can occur in subgrade materials. In many ways, characterization of the subgrade materials is very similar to characterization of unbound granular; i.e., the material must be characterized with respect to a state of stress, moisture condition, and material density. As with the unbound base, the repetitive triaxial test is considered to be the appropriate laboratory test for characterizing the material. For the cohesionless subgrade, the characterization procedure is the same as for the unbound base, except the material quality and material densities will not meet the requirements for base materials; thus, some of the appropriation procedures will not be valid. For cohesive soils, the modulus is more a function of the deviator stress than the first stress invariant; hence, this is the stress parameter used in characterization of the cohesive subgrades. Based on the stress study, a 34.5-kPa (5-lbf/in.^{2}) deviator stress is considered to be an adequate estimation for design of a typical airport pavement. As with the bases, since these values of stress are estimates for a typical pavement, it may be desirable to check these estimates through computation of the stresses before the design is finalized.

The moisture content at which the laboratory tests are conducted is very critical. Studies of the moisture content in airport subgrades have indicated that in most cases the subgrade material is near saturation. In selecting a moisture content at which the laboratory tests are to be conducted, the final or equilibrium water content should be considered. In some design situations, the designer may have information indicating that this final moisture content would be less than saturated. For such cases, savings may be realized by designing the pavement on the expected subgrade moisture content rather than the saturated water content.

Much effort has been applied toward developing correlations between other material parameters and resilient modulus. A study was conducted by the U. S. Army Engineer Waterways Experiment Station for correlating modulus of subgrade reaction as determined by the plate bearing test. Such correlations can be useful in estimating the resilient modulus of the subgrade, but as with any empirical correlation care must be exercised in their use. Particular care must be taken in the case of field tests conducted to ensure that the moisture conditions at the time the tests are conducted will be representative of the final water content of the subgrade.

When the moisture content of the subgrade approaches the saturation moisture content, the Poisson's ratio of the material will approach 0.5. In the study at the Waterways Experiment Station, 0.4 has been found to be a representative value for most subgrades. For subgrades that are to be considered near saturation, it is suggested that 0.4 would be an adequate estimation of Poisson's ratio. If some lesser moisture content is considered applicable for design, then Poisson's ratio should be measured, or some lesser value of Poisson's ratio should be appropriate.

Summary for Material Characterization

Table 1 provides a summary for determining the modulus and Poisson's ratio for use in the analysis of rigid pavements. The column entitled "Influencing Parameters" may need additional explanation since the parameters listed are by no means all of the parameters that influence the modulus of a material. First, the table assumes that the material tested is used in the pavement and meets the material specifications. For example, for the bituminous concrete, it is assumed that the aggregate, bitumen, moisture, and density are the same as to be used and do meet the specifications. Thus, in setting up the laboratory tests many influencing parameters will be fixed but others, i.e., those parameters listed, will be

Table 1. Summary chart for material characterization.

Material	Referenced Laboratory Test	Influencing Parameters	Expected Range for Modulus	Other Methods of Estimating Modulus by Correlations	Typical Values or Range for Poisson's Ratio
PCC	Flexural beam	Age	3×10^6 - 6×10^6	Split tensile Unconfined compression	0.2
Bituminous concrete	Flexural beam	Temperature, rate of loading	1×10^5 - 1.5×10^6	Split tensile Triaxial compression	0.3-0.5
Granular base	Triaxial compression	First stress invariant, density, moisture content	1×10^4 - 6×10^4	Empirical charts	0.3
Cohesionless subgrade	Triaxial compression	First stress invariant, density moisture content	1×10^4 - 3×10^4	Plate bearing	0.3-0.5
Cohesive subgrade	Triaxial compression	Deviator stress, density, moisture content	4×10^3 - 3×10^4	Plate bearing	0.4

variable, and particular care must be exercised in the control of these test parameters.

Development of Performance Criteria

Basis for Data

Historically, performance criteria for rigid pavements have been based on limiting the tensile stress in the PCC slab. Cracking or other forms of pavement distress are attributed to the repeated application of loads, and the process and criteria are referred to as fatigue and fatigue criteria. The fatigue process for rigid pavements is assumed to be similar to that for a PCC beam, and the criteria are presented in the same manner as the results of fatigue testing of concrete beams; i.e., a plot is made of the ratio of the applied stress to the strength of the PCC versus the number of stress repetitions applied.

For the performance criteria for the design procedure, the basic data were developed from test pavements subjected to simulated aircraft traffic. The term coverages will serve as the measure of traffic or stress repetitions. The elastic layered idealization and in particular the BISAR computer code will compute the limiting stress. The maximum principal tensile stress occurring in the bottom of the PCC slab was selected as the critical response parameter. The initial crack definition of failure selected stated that a pavement was considered failed when approximately 50 percent of the slabs in the traffic area had cracked. No allowances were made for satisfactory performance after initial cracking for pavements on high-strength foundations.

Pavement Test Data

The pavements from which the performance criteria were developed are described in Reference 2 with summary in Table 2. These pavements were constructed and tested by the Corps of Engineers from 1943 to 1973. The procedures followed in each of the test tracks were basically the same. The pavement sections were designed to answer specific questions or to solve certain problems; the pavements were constructed and the as-constructed properties measured; the type of traffic needed to answer the specific questions or to solve the specific problems was applied to the pavement; the volume of traffic and pavement condition were monitored until failure of the pavement was achieved; and the after-traffic properties of the pavement were measured. From these tests, 60 data points were developed.

The use of data from actual traffic tests permits a number of factors to be considered indirectly in the design. The performance is directly related to traffic, and the resulting criteria are for the entire pavement rather than for one component or material, such as fatigue of the PCC as obtained from beam tests. Environmental effects are not considered in the computation of the response parameters for developing the criteria. Although the full-scale pavements experienced the effects of temperature and moisture changes, gradients, frictional restraint forces, etc., the range of conditions experienced was not all-inclusive.

The effects of joints are not considered directly although it is recognized that joints are a point of weakness in rigid pavements and the distress is usually initiated at joints. The effects of joints are handled in the following manner:

1. The test pavements all had joints and slab sizes that were similar to the standard types used in airfield pavements.
2. The traffic was applied in the most critical manner with respect to the joints.
3. The use of the criteria will result in adequate pavements provided that the standard joint types and similar slab sizes used ensure adequate vertical forces transmitted between slabs.

In the development of the criteria, it was necessary that properties be assigned to the materials of

Table 2. Data for development of performance criteria.

Site Location	Item Identification	Type Load	Traffic, Coverages	Design Factor	Vertical Deflection (mm)
Lockbourne No. 1	A	Dual-Wheel	390[ab]	1.84	0.52
	A		45	1.31	0.90
	B		187	1.47	0.82
	B		35	1.03	1.42
	C		200	1.33	1.11
	C		44	0.92	1.92
	D		450	1.30	1.26
	D		33[a]	0.89	2.19
	E		430[a]	1.48	1.04
	E		77[ab]	1.02	1.81
	F		550[ab]	1.87	1.13
	F		111	1.25	1.96
	K		72	1.29	1.61
	K		700	1.91	1.00
	N		150	1.39	1.64
	N		9	0.94	2.67
	O		573	1.70	1.46
	O		72	1.14	2.36
	P		262	1.24	2.27
	P		6	0.84	3.66
	Q		1,390	1.68	1.67
	Q		57	1.12	2.69
	U		88	1.51	1.02
	U		1.5	1.16	1.59
	A (Reconstr.)	Dual-Wheel	658	1.87	1.10
Lockbourne No. 2	E1	Single-wheel	97	1.15	1.63
	E2		942	1.19	1.48
	E3		17	1.07	2.05
	E4		203	1.07	2.00
	E5		43[a]	1.53	1.22
	E6		2,204[a]	1.77	1.03
	E7	Single-Wheel	2,204[ab]	2.44	0.88
	M1	Dual-Tandem	134[a]	1.21	2.64
	M2		2,204[a]	1.63	2.05
	M3	Dual-Tandem	2,204[ab]	2.46	1.43
Lockbourne No. 3	1	Dual-Wheel	18	0.82	2.29
Sharonville Channelized	57	Dual-Wheel	34,650[ab]	2.35	1.02
	58		34,650[a]	1.98	1.12
	59		7,600	1.86	0.76
	60		1,674	1.80	0.46
	61		3,867	2.13	0.43
	62	Dual-Wheel	10,082	2.72	0.33
Sharonville Heavy Load	71	Dual-Tandem	9,680[ab]	3.28	0.75
	72		9,680	2.53	1.05
	73	Dual-Tandem	2,115	2.00	1.29
MWHGL	1	C-5A	221	1.25	1.47
	2	C-5A	4,230	1.69	1.28
	2	Dual-Tandem	95	1.24	1.37
	3	Dual-Tandem	205	1.43	1.15
	3	C-5A	1,400	1.78	1.10
	4	C-5A	180	1.05	1.73
KLJS	1	C-5A	54	1.39	1.50
	2		344	1.40	1.35
	3		22	1.40	1.47
	4	C-5A	6,336	1.76	1.47
	4	Dual-Tandem	320	1.43	1.50
SSPS	3	Dual-Tandem	3,215	2.00	1.21
	3		350	1.62	1.47
	4		4,660	2.00	1.21
	4	Dual-Tandem	70	1.61	1.46

Note: 1 mm = 0.039 in.

[a]The plus sign (+) indicates that traffic stopped prior to item failure.

[b]Data points not used in analysis.

each pavement section. For most of the test sections, the subgrade had been characterized by material description and the modulus of subgrade reaction as determined by the static plate bearing test. For these subgrades, the modulus of elasticity was obtained from the correlation between the modulus of elasticity and the static modulus of soil reaction.

The moduli of elasticity for the granular base materials in the earlier test sections were selected using data from correlation studies, combined with information from other sources. This additional information was obtained from procedures for determining moduli of granular materials described by Barker and Brabston (1). The flexural strengths of the PCC slabs were provided in the data reports for the tests.

Limiting Stress Criteria

The limiting stress (fatigue) criteria were developed as a relationship between design factor and coverages. The basic definitions of coverages and design factor are the same as used previously for the Corps of Engineers criteria. The one difference in detail is that stress as computed with the elastic layered theory is substituted for the stress computed with the Westergaard idealization, and the factor to account for stress reduction due to shear between slabs is omitted. Design factor is now defined as

$$DF = R/\sigma \qquad (1)$$

in which DF = design factor for stress computed with the elastic layered theory; R = PCC flexural strength; and σ = maximum principal tensile stress at bottom of PCC slab computed with elastic layered model.

The limiting stress criteria are illustrated in Figure 5. The complete data for this figure are provided in Reference 2 and Table 2.

Several curve fitting techniques were tried on these data points. The one finally selected was a linear relationship fitted to the data by means of least squares with the design factor as the dependent variable and the logarithm of the coverages as the independent variable. The resulting relationship in Figure 5 is described by the equation

$$DF = 0.58901 + 0.35486 \log(Cov) \qquad (2)$$

where log(Cov) = the base 10 logarithm of the traffic in terms of coverages.

The absence of data points for traffic volumes greater than 10,000 coverages should be noted. Only one point is shown for coverage levels greater than 10,000, and this pavement did not reach failure at the indicated traffic level. Use of the relationship to design for traffic volumes greater than 10,000 coverages (which will frequently be the case for current traffic volumes) will require extrapolation of the linear relationship.

Attempts were made at correlating both surface deflection and subgrade strain to pavement performance, but no correlations could be made that improve performance predictions. The relationships between these two parameters and performance appear to be greatly influenced by aircraft gear characteristics and subgrade stiffness.

Comparison of Criteria

In working through the design examples, it was obvious that slab thicknesses obtained by applying stress criteria (equation 2) were different from the thicknesses obtained from present Corps design procedures. For stress criteria, the magnitude of the difference was unexpected and felt to be unacceptable. To understand the nature of the differences in criteria, additional analyses were conducted.

First, the required slab thicknesses for different design situations were computed using each of the design criteria. Figures 6 and 7 present the comparisons of the required slab thicknesses over a range of subgrade strengths for DC-8 and C-141 aircraft. The stress criteria appeared, at least for higher levels of traffic, to give slab thicknesses that were significantly greater than those determined from present design criteria.

For additional comparison between stress criteria and present design criteria, coverage predictions (Table 3) were made for test sections using both criteria. Figure 8 shows the comparison of the two criteria. These data are evidence that the two criteria are different even though much of the same test data were used in development of both criteria.

The difference is particularly noticeable at lower coverages (below 300) and higher coverages (from 1,000 to 30,000). Figure 9 presents a comparison of both criteria with actual coverages to failure. The comparison of Figure 9 clearly shows that stress criteria as developed in this study better predict the actual performance, at least for lower and higher coverages, than the present design criteria.

To better understand why present design criteria are failing to predict performance in certain coverage ranges, test data were reanalyzed in the same manner; i.e., the plate theory was used to compute edge stresses, as was done to develop the original Corps design criteria. Also, present design criteria were used to develop interior stress criteria of the form of equation 2. This was accomplished by designing, based on present design criteria, a number of pavements covering a range of design conditions. The design factors for these hypothetical sections were computed in the same manner as the design factors for the test sections. The result was interior stress criteria, which best fit the present design criteria.

Figure 10 shows the comparison between present design criteria and data developed from the test sections. It should be repeated that these data shown were computed using edge stresses in the same way as was used to develop the present design criteria. Specific reasons for disparity between the criteria and the test section data are not known. One contributing factor may have been that a considerable amount of data shown is from test sections constructed and tested after the development of the present design criteria. The lack of data would not completely explain the disparity, but no other explanations can be offered. Figure 11 provides a comparison of stress criteria (equation 2) with the present design criteria. From this comparison, it is clear that for low coverages the new stress criteria will require pavements of lesser thicknesses and for high coverages will require pavements of greater thicknesses.

The conclusion from the comparisons of the two criteria is that a basic difference does exist between the two criteria. Test section data as analyzed in this study definitely support new criteria, and acceptance of new criteria is justified.

Closure

The basic design procedure has been completed and efforts are under way by the Corps and the Federal Aviation Administration to implement the procedure.

Figure 5. Design criteria.

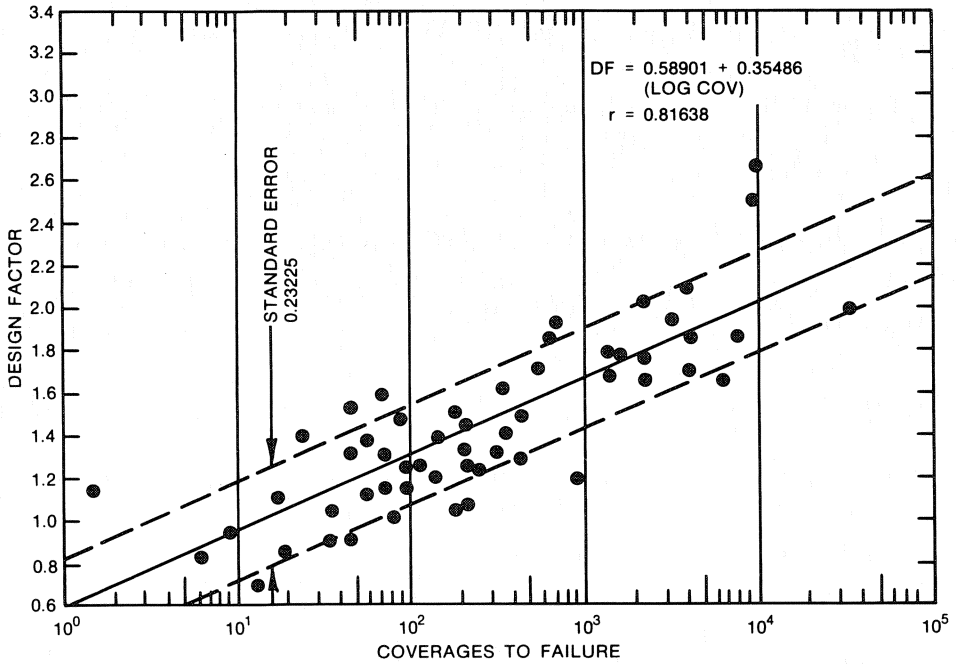
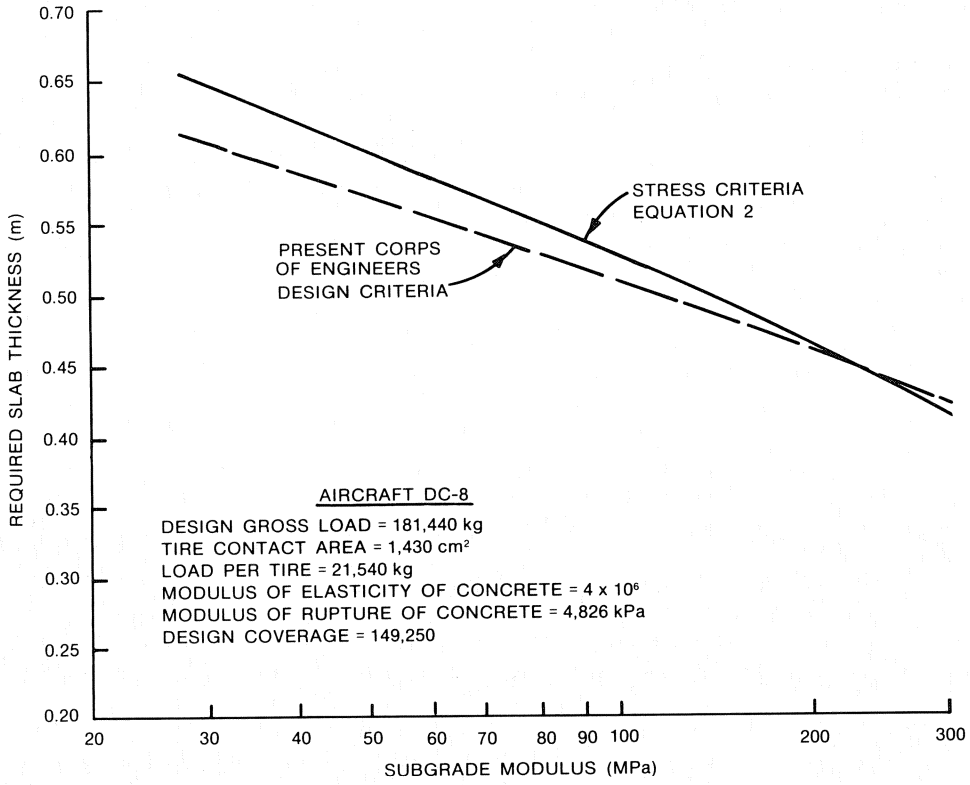
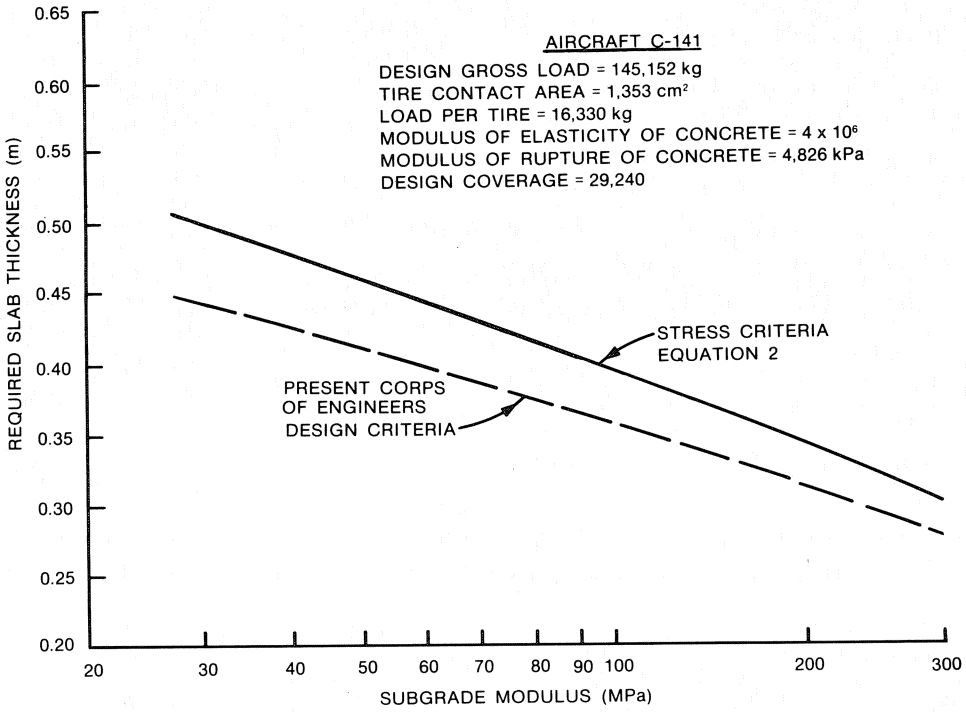


Figure 6. Comparison of stress criteria with present design criteria - DC-8 aircraft.



Note: 1 m = 39.37 in.; 1 MPa = 145 kips/in.^{2}; 1 kg = 0.002 kip; 1 cm^2 = 0.155 in.^{2}; 1 kPa = 0.145 lbf/in.^{2}.

Figure 7. Comparison of stress criteria with present design criteria - C-141 aircraft.



Note: 1 m = 39.37 in.; 1 MPa = 145 kips/in.^{2}; 1 kg = 0.002 kip; 1 cm^2 = 0.155 in.^{2}; 1 kPa = 0.145 lbf/in.^{2}.

Figure 8. Comparison of coverage predicted by present design criteria with coverage predicted by stress criteria.

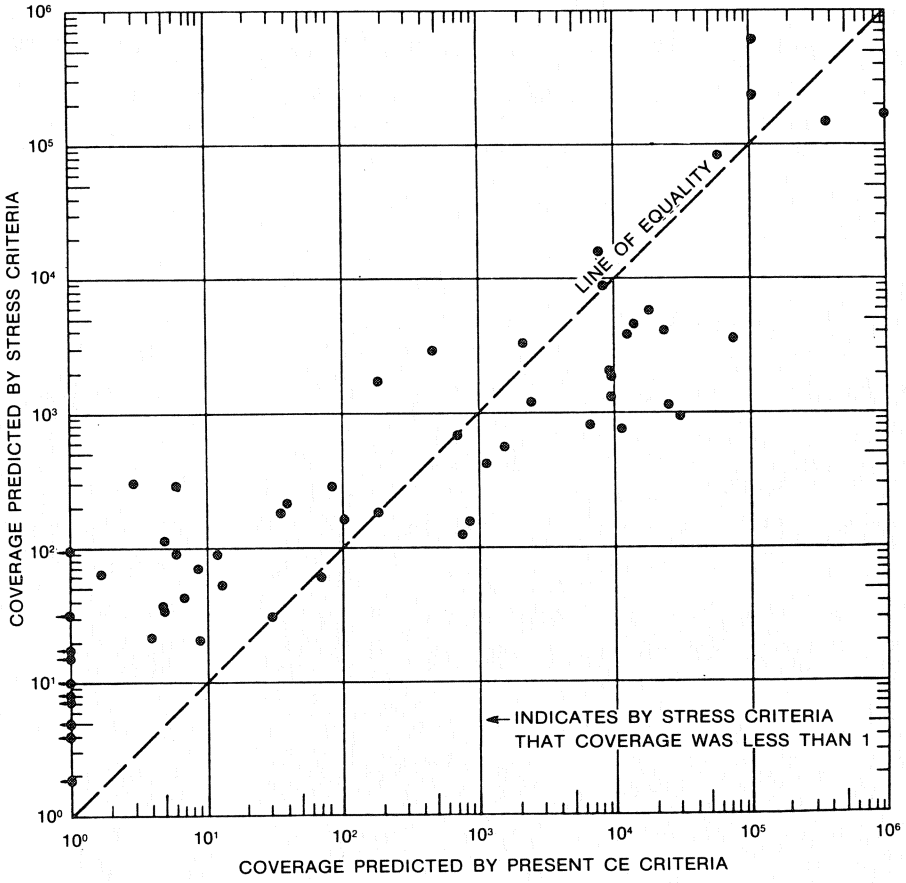


Table 3. Data for comparison of stress and present criteria.

Test Series	Test	Thickness (cm)	k[a] (kPa/cm)	R[b] (MPa)	0.75 σ_e[c] (MPa)	H[d] (%)	σ_{LT}[e] (MPa)	Coverages Actual[f]	Coverages Present[g]	Coverages Equation 2[h]
Lockbourne No. 1	A-1	14.5	407	5.10	4.32	92.7	2.79	390.0+	461.000	7,888
	A-2	14.5	407	5.38	6.28	71.3	4.13	45.0	0.400	96
	B-1	14.0	204	5.10	5.35	79.4	3.47	187.0	6.000	286
	B-2	14.0	204	5.38	7.89	60.6	5.23	35.0	0.010	17
	C-1	14.0	190	5.10	5.43	78.6	3.85	200.0	5.000	116
	C-2	14.0	190	5.38	8.03	59.8	5.88	44.0	0.010	8
	D-1	14.0	204	5.10	5.35	79.4	3.94	450.0	6.000	90
	D-2	14.0	204	5.38	7.89	60.6	6.05	33.0	0.010	7
	E-1	14.6	282	5.10	4.69	87.5	3.48	430.0+	85.000	286
	E-2	14.6	282	5.38	6.88	66.9	5.32	77.0	0.100	15
	F-1	19.7	141	5.10	3.62	106.0	2.73	550.0+	12,086.000	3,734
	F-2	19.7	141	5.38	5.54	80.6	4.31	110.0	9.000	70
	K-3	24.0	244	5.07	5.03	81.4	3.93	72.0	12.000	90
	K-2	24.0	244	5.38	3.77	107.3	2.83	700.0	14,606.000	4,527
	N-2	20.3	204	5.38	4.89	88.1	3.89	150.0	103.000	161
	N-3	20.5	204	5.07	6.45	66.7	5.41	9.0	0.100	10
	O-2	24.0	204	5.38	3.92	104.2	3.16	573.0	9,265.000	1,253
	O-3	24.0	204	5.07	5.27	78.9	4.46	72.0	5.000	34
	P-2	19.3	258	5.38	4.97	86.9	4.36	262.0	69.000	61
	P-3	19.3	258	5.07	6.46	66.0	6.09	6.0	0.100	5
	Q-2	24.0	296	5.38	3.61	110.7	3.21	1,390.0	24,064.000	1,102
	Q-3	24.0	296	5.07	4.78	84.4	4.54	57.0	31.000	10
	U-2	14.8	562	5.38	5.64	77.4	3.63	88.0	3.000	305
	U-3	14.8	562	5.07	6.86	60.0	4.49	1.5	0.100	32
	A-REC	24.9	296	5.00	3.11	118.3	2.69	658.0	73,462.000	3,102
Lockbourne No. 2	E-1	38.1	244	5.00	5.49	78.5	4.34	97.0	5.000	37
	E-2	38.1	407	4.69	4.97	79.8	3.96	942.0	7.000	44
	E-3	38.1	269	4.90	5.39	78.2	4.57	17.0	4.000	22
	E-4	38.1	445	4.69	4.87	80.7	4.43	203.0	9.000	21
	E-5	47.6	247	4.79	3.95	95.5	3.13	43.0	1,149.000	421
	E-6	57.5	263	4.83	3.48	104.4	2.74	2,204.0+	9,541.000	1,843
	E-7	61.0	269	5.24	2.74	129.2	2.15	2,204.0+	364,000.000	145,115
	M-1	30.5	149	5.00	4.96	81.7	4.14	134.0	13.000	54
	M-2	38.1	149	5.00	3.74	102.1	3.08	2,204.0+	6,806.000	800
	M-3	50.9	149	5.00	2.51	136.1	2.06	2,204.0+	1,003,000.000	165,000
Lockbourne No. 3		15.2	168	5.52	9.23	57.3	6.73	18.0	0.004	4
Sharonville Channelized	57	50.8	73	5.10	3.08	116.6	2.17	34,650.0	57,233.000	81,420
	58	45.7	81	5.10	3.58	105.8	2.57	34,650.0+	11,718.000	7,567
	59	40.6	128	5.03	4.03	97.4	2.72	7,600.0	2,136.000	3,284
	60	30.5	909	5.03	4.51	89.8	2.87	1,674.0	179.000	1,728
	61	35.6	814	5.03	3.71	103.1	2.41	3,867.0	7,883.000	15,335
	62	40.6	977	5.03	2.95	120.9	1.89	10,082.0	107,620.000	595,949
Sharonville Heavy	71	81.3	271	5.52	2.48	145.4	1.72	9,680.0+	3,900,000.000	20,368,141
	72	71.1	190	5.52	3.21	120.9	2.20	9,680.0	107,620.000	227,465
	73	61.0	190	5.52	4.03	103.6	2.76	2,115.0	8,483.000	8,604
MWHGL	1-C5	25.4	163	5.00	5.65	73.6	4.00	221.0	0.910	70
	2-C5	30.5	190	5.52	4.36	97.8	3.26	4,230.0	2,433.000	1,176
	3-C5	35.6	201	4.83	3.52	104.1	2.72	1,400.0	9,130.000	2,095
	4-C5	20.3	201	5.34	6.99	64.3	5.07	180.0	0.040	19
	2-DT	30.5	190	4.82	5.37	75.5	2.90	95.0	1.700	65
	3-DT	35.6	201	4.55	4.39	85.2	3.18	205.0	40.000	221
KLJS	1-C5	20.3	679	6.24	5.15	94.7	4.52	54.0	885.000	160
	2-C5	27.9	271	5.03	4.42	89.9	3.60	344.0	185.000	182
	3-C5	25.4	217	5.58	5.26	84.8	4.00	22.0	35.000	182
	4-C5	25.4	638	5.93	3.97	112.1	3.60	6,336.0	29,557.000	909
	4-DT	25.4	638	5.93	4.89	94.2	4.43	320.0	752.000	124
SSPS	3-200	38.1	326	6.21	4.28	108.9	3.19	3,215.0	18,474.000	5,853
	3-240	38.1	326	6.21	5.14	94.0	3.89	350.0	704.000	659
	4-200	38.1	407	4.00	4.05	110.5	3.19	4,660.0	23,368.000	3,982
	4-240	38.1	407	6.00	4.87	96.0	3.83	70.0	1,353.000	544

Note: 1 cm = 0.3937 in.; 1 kPa/cm = 0.369 lbf/in.^{3}; 1 MPa = 145 lbf/in.^{2}.

[a] k = the modulus of subgrade reaction.

[b] R = the modulus of rupture for the concrete slab.

[c] σ_e = the edge stress as computed by plate theory.

[d] H = the percent standard thickness computed for development of design criteria.

[e] σ_{LT} = the interior stress as computed by layer theory.

[f] Actual = actual coverage to failure or if a + is used would mean section did not fail and the coverage given is when traffic was terminated.

[g] Present = the predicted coverage to failure as determined from present design criteria.

[h] Equation 2 = the predicted coverage to failure as determined from the design criteria developed in this study.

Figure 9. Comparison of actual coverage to failure with predicted coverage to failure.

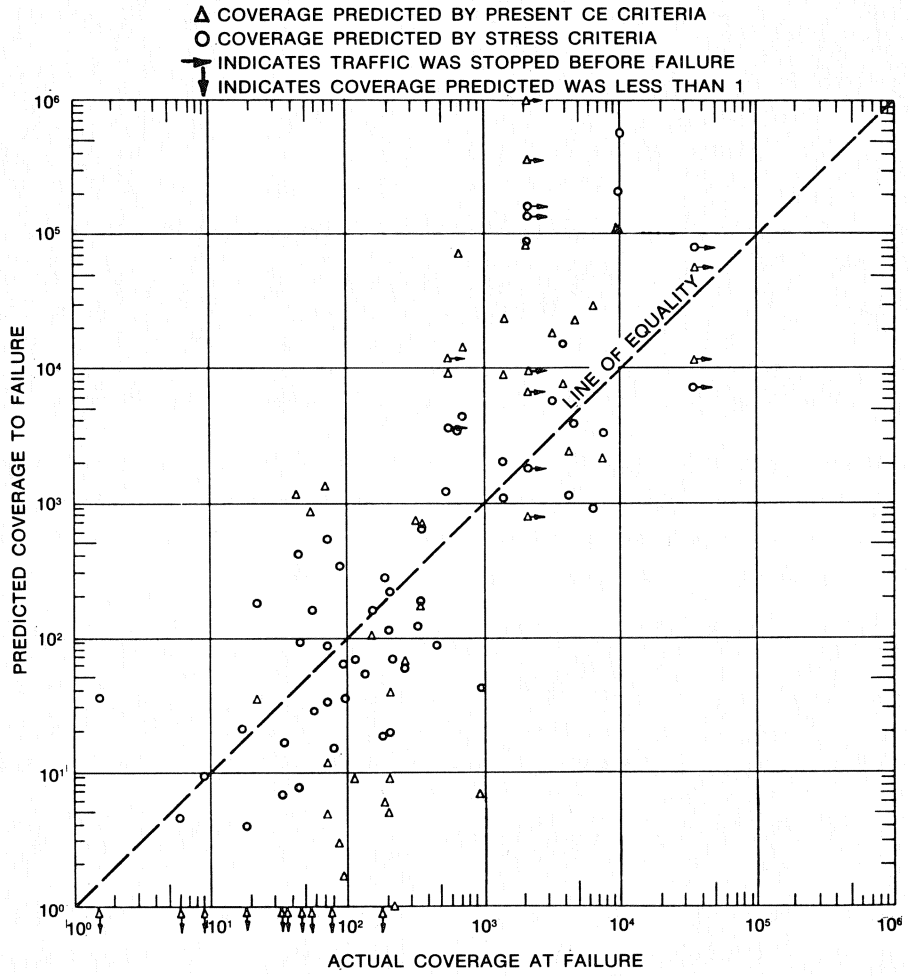


Figure 10. Comparison of present design criteria with test section data.

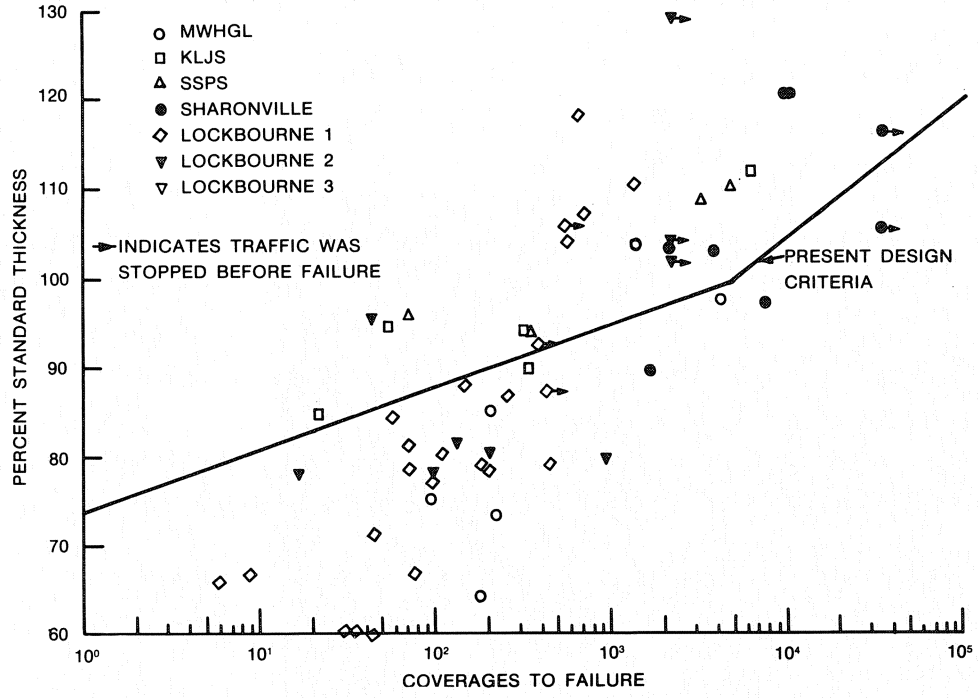
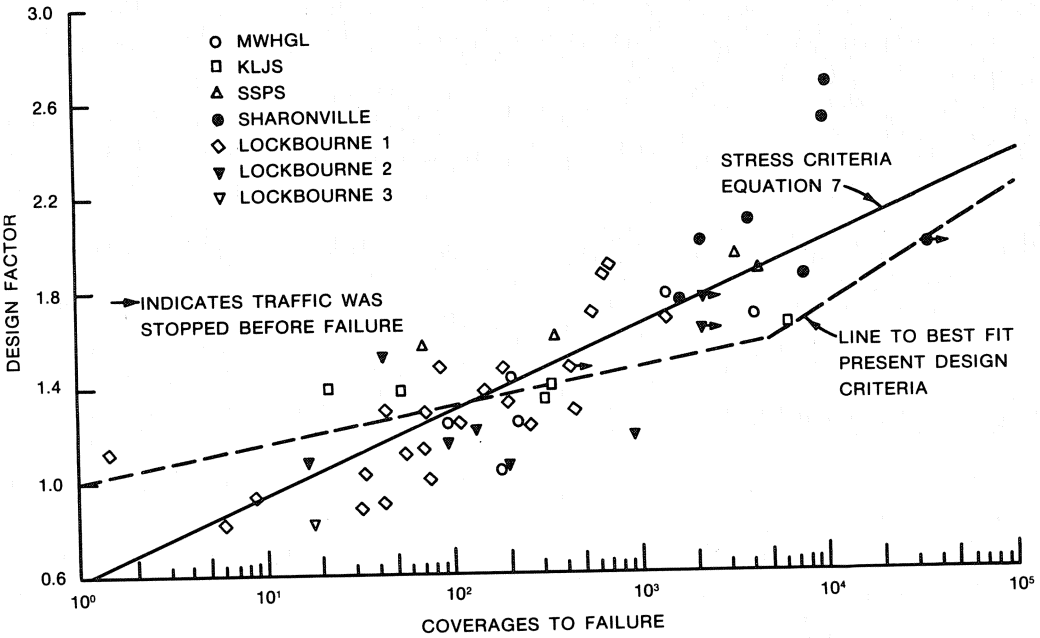


Figure 11. Comparison of stress (equation 2) with present design criteria.



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