MECHANISTIC BEHAVIOR OF PAVEMENT SYSTEMS

HAND-OUT NOTES FOR A
SEMINAR SERIES PRESENTED

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MECHANISTIC BEHAVIOR OF PAVEMENT SYSTEMS

Reference Text: "Principles of Pavement Design", 2nd. ed., by E.J. Yoder and M.W. Witczak, Wiley, 1975

OUTLINE

PREFACE

As part of their continuing effort to prolong the life of existing pavement systems, highway engineers must continue to improve upon design and rehabilitation methods. In past years, design methods based upon empericism have been employed with considerable success. As loading and materials change or become more complex, the older practices become less useful and a new approach undertaken. Throughout the U.S. and elsewhere, there has been emerging new technology often termed "rational", "systematic", "mechanistic", and other terms used to describe new approaches to pavement analysis and design. Most of these are based on elastic or viscoelastic layered representation of the pavement structure and have the potential to become powerful tools for the highway engineer.

In recent years, the University of Washington has assisted the Washington State Highway Department on various research projects. One project resulted in a report* that has been used to develop guidelines for allowable truck traffic through the concept of pavement damage and/or remaining life. The basis for this report was the concept that pavement structures can be reasonably well represented by elastic layered systems. Analysis of the mechanistic behavior can then be used to predict pavement response and life expectancy.

As a result of this study and after discussions with Highway Department personnel, it was determined that the procedures, techniques, as well as other information used in the above report may be of further interest to them. In other words, the methods as well as results had potential utilization in a wide range of applications. As a result, a seminar program was set up whereby the writer would conduct or lead weekly discussion sessions on key topics.

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* "Pavement Response and Equivalencies for Various Truck Axle-Tire Configurations" by Ronald L. Terrel and Sveng Rimsritong, WSHD Research Report 17.1 November, 1974.

The notes included herein are a result of a need for hand-out material and are not necessarily intended as a text, but served as reference material to the oral presentations.

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"THE PAVEMENT AS A LAYERED SYSTEM"

Pavements could be easily classified into two main categories, namely Flexible and Rigid Pavements. Rigid pavements (PCC concrete) are made up of portland cement concrete and may or may not have a base course between the pavement and the subgrade. Flexible pavements are pavements having a relatively thin asphalt wearing course with layers of granular base and subbase, used to protect the subgrade from being overstressed. In order to increase the structural strength of the pavement, stabilizers such as asphalt, cement, lime, flyash and chemicals are sometimes added to the base and subbase material.

The design and construction of pavements had been primarily based on empiricism or experience with theory only playing a subordinate role. The changes resulting from heavier wheel loads, higher traffic levels and the recognition of various independent distress modes contributing to pavement failures have led to the search for a more rational method of pavement design in the last several years. The use of multilayered elastic theory in pavement design is gaining ground as this concept is being incorporated into several design procedures.

ELASTIC LAYERED SYSTEMS:

The response of pavement systems to wheel loadings has been of interest since 1926 when Westergaard (1) used elastic layered theory to predict the response of rigid pavements. Later Burmister (2) solved the problem of elastic multilayered pavement structure using classical theory of elasticity. The assumptions that Burmister and most others have made to the state of stress or strain are as follows:

1. Each layer acts as a continuous, isotropic, homogeneous, linearly elastic medium;
2. Each layer has a finite thickness except for the lower layer, and all are infinite in the horizontal directions;

3. The surface loading can be represented by a uniformly distributed vertical stress over a circular area;
4. The interface conditions between layers can be represented as either perfectly smooth or perfectly rough;
5. Inertial forces are negligible;
6. Deformations throughout the system are small;
7. Temperature effects are negligible except for bituminous treated layers;
8. The stress solutions are characterized by two material properties for each layer - Poisson's ratio μ, and the elastic modulus E.

Figure 1 illustrates the general concept of a multilayered elastic system.

One Layer Systems (3):

The analysis of stresses, strains and deflections have been primarily derived from the Boussinesq equation originally developed for an homogeneous, isotropic, and elestic media due to a point load at the surface.

According to Boussinesq, the vertical stress at any depth below the earth's surface due to a point load at the surface is given by

$$\sigma z = K \, {}^{P}/_{Z^2}$$

$$K = \frac{3}{2\pi} \, \frac{1}{\left[1 + (r/z)^2\right]^{5/2}}$$

where r = radial distance from point load

Z = depth.

Here the vertical stress is dependent on the depth and radial distance and is independent of the properties of the transmitting medium. The distribution of vertical stress below a concentrated load on any horizontal plane takes the form of a bell-shaped surface; maximum stresses occurring on the vertical plane passing through the point of application.

In actual flexible pavements, the load at the surface is not a point load but is distributed over an elliptical area. Pressures at the tire-pavement contact are equal to the tire pressure. Variation of stress with depth follows the same general pattern as for point-load case.

Several influence charts and tables have been developed to determine the stresses, strains and deflection at any point in a homogeneous mass for any value of Poisson's ratio. Table 2.1 and Table 2.2 (Yoder and Witczak) give solutions for the various parameters. Although most asphalt pavement structures cannot be regarded as being homogeneous, the use of the equations in Table 2.1 are generally applicable for subgrade stress, strain, and deflection studies when the modular ratio of the pavement and subgrade is close to unity.

In one layer theory, the pavement portion above the subgrade does not contribute any partial deflection to the total surface deflection; thus

$$\Delta_T = \Delta p + \Delta s$$

where Δ_T = total surface deflection

Δp = deflection within the pavement layer = 0

Δs = deflection within the subgrade layer

Two Layer Systems

All typical flexible pavements are composed of layers in which the moduli of elasticity decrease with depth. This effect reduces the stresses and deflections in the subgrade from those obtained for the ideal homogeneous case.

Solutions to two layer problems have been obtained by Burmister. Stress and deflection values are dependent on the strength ratio of the layers, E_1/E_2 where E_1 and E_2 are the moduli of the reinforcing and subgrade layers respectively. Figure 2.6 (Yoder and Witczak) shows the vertical stress values under the center of a circular plate for the two-layer system.

For a two-layer system, the total surface deflections can be obtained from the following:

Flexible Plate: $\Delta = 1.5 \frac{pa}{E_2} F_2$

Rigid Plate: $\Delta = 1.18 \frac{pa}{E_2} F_2$

where p = unit load on circular plate

a = radius of plate

E_2= modules of elasticity of lower layer

F_2= Dimensionless factor depending on the ratio of moduli of elasticity of the subgrade and pavement as well as the depth of radius ratio.

Curves of F_2 for various depth ratio and E are shown on Fig. 2.7 (Yoder and Witczak).

Interface deflection factor charts have also been developed by Huang as an extension of Burmister's 2 - layer theory. These F factors are given in Fig. 2.8 (Yoder and Witczak) and the interface deflection Δs is given by

$$\Delta s = \frac{pa}{E_2} F$$

F is a function of pavement depth or thickness and offset distances expressed in radii.

Three Layer System

For a 3 - layer pavement we can have the stress situtaiton represented by the following -

σz_1 and σz_2 Vertical stresses at interface 1 and 2

σr_1 and σr_2 Horizontal stresses at the bottom of layer 1 and 2

σr_3 Horizontal stress at the top of layer 3

Vertical stress solutions have been obtained by Peattie and are shown in graphical form in Fig. 2.10 (Yoder and Witczak).

Horizontal stress solutions have been obtained by Jones and are shown in Tabular form in Table 2.3 (Yoder and Witczak). The figures and tables have all been developed for $\mu = 0.5$ for all layers.

Both the vertical stress (graphical solutions) and the horizontal stress (tabular solutions) use the following parameters.

k_1 or $K1 = \frac{E_1}{E_2}$ k_2 or $K2 = E_2/E_1$

a_1 or $A = a/h_2$ $H = \frac{h_1}{h_2}$

Several combination values for these parameters are presented. From Figure 2.10, stress factor values ZZ1 or ZZ2 are for particular K1, K2, A and H values of the pavement system. The vertical stresses are then $\sigma z_1 = p(ZZ1)$

$$\sigma z_2 = p(ZZ2)$$

From Table 2.3, for a particular combination of input parameter K1, K2, a. and H, the stress factors are given in stress difference, ie (ZZ1 - RR1), (ZZ2 - RR2), and (ZZ2 - RR3). The horizontal stresses can be calculated from

$$\sigma z_1 - \sigma r_1 = p[ZZ1 - RR1]$$

$$\sigma z_2 - \sigma r_2 = p[ZZ2 - RR2]$$

$$\sigma z_2 - \sigma r_2 = p[ZZ2 - RR3]$$

Several types of strains can also be calculated. For example, knowing σz_1 and σr_1, the horizontal strain at the bottom of layer 1, εr will be computed from

$$\varepsilon r_1 = \frac{\sigma r_1}{E_1} - \mu_1 \frac{\sigma t_1}{E_1} - \mu_1 \frac{\sigma z_1}{E_1}$$

Note that in all the analysis $\sigma t_1 = \sigma r_1$ and for $\mu = 0.5$

$$\varepsilon r_1 = \frac{1}{2E_1} (\sigma r_1 - \sigma z_1)$$

Two and three layer systems may be analyzed by hand methods. If there are more than three layers, similar layers can be grouped together and assigned average physical properties, in order to reduce the number of layers to three. The use of tables and charts can be quite tedious and time consuming for 3 - layer pavement systems, and tabulated solutions of 4 - layer systems are not practical. Therefore, systems with more than two layers are most conveniently analyzed by computer methods.

LIMITATIONS OF LAYERED THEORY (4)

1. In Classical layered theories, both load and pavement geometrics are symmetrical about a common centerline (axisymmetry). Unfortunately, the effects of wheel loads applied close to a crack or pavement edge cannot be analyzed by the use of methods which require axisymmetry.

2. Information is not available on the conditions of slip which exists at the interface between layers. The assumption of a rough interface condition, which most investigators have used (5,6) appears to be reasonable, although varying degrees of slip can be considered.

3. In all of the theoretical approaches, inertial forces have been neglected. Also, none considers the effects of vibrations. Neglecting vibrations is probably not a bad assumption for vehicle speeds lower than 60 mph (4) on materials having cohesion. However, for cohesionless materials compacted to lower relative densities, neglecting vibratory effects may lead to densification that would cause rutting and changes in material properties.

4. Laboratory tests have indicated that the dynamic modulus of paving materials varies with the confining pressure or deviator stress or both (7). Because of the variation in stress state that exists in each layer of the pavement system, the dynamic modulus actually changes with both depth and lateral position in each layer. Therefore, uncertainties arise in trying to determine what values of dynamic modulus to use in a linear elastic layered analysis. Further, elastic layered theory cannot consider variations in the modulus with lateral position. Those limitations for the most part would have been overcome by the use of nonlinear finite - element theory (8).

5. The bottom of each layer of a pavement structure is subjected to radial and tangential tensile stresses and strains. In nonstabilized granular materials, the application of such a stress state can result in an appreciable influence on the modulus of the material. The behavior of granular materials under these stress conditions certainly need further study.

6. Although most pavement materials are nonlinear, the use of a linear model will suffice provided the stress states are low.

DESIGN IMPLICATIONS

Presently several agencies are adopting the use of elastic layered theory in the design and evaluation of pavement systems (9,10,11,12). Shell (9) has incoporated fatigue in the design of highway pavements since 1963. The criterion developed by Shell has also been used extensively since 1963 in the design and evaluation of pavement systems subjected to unusual wheel loads. The Asphalt Institute (10) followed using similar, yet more sophisticated, tools to develop a precedure to design and evaluate airfield pavements to account for jumbo jet operations.

The Kentucky Highway Department has also developed a design procedure using elastic layered theory. The Chevron program (5) was used in this procedure to calculate stresses and deformations in the pavement system and design criteria were developed based on observed field performance.

More recently, Chevron Research (12) has developed a "Simplified Thickness Design Procedure for Asphalt and Emulsified Asphalt Pavements". The procedure is based on analysis of layered systems (5) and considers only 2 - layered systems (full-depth design plus subgrade). Critical strains in the pavement system are limited to values depending on expected service life. Thicknesses are then determined to minimize the amount of permanent deformation and/or fatigue cracking.

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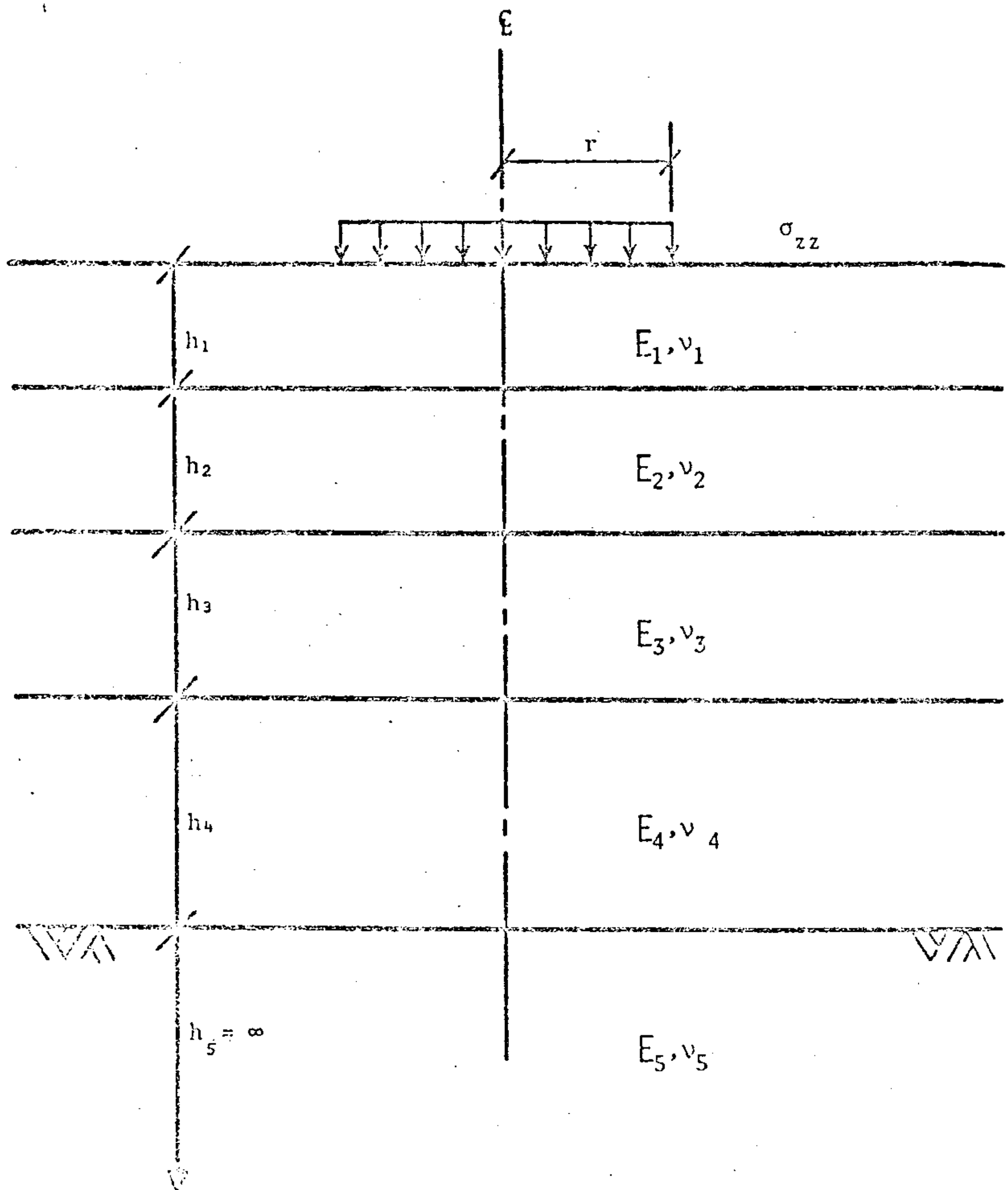


Fig. 1(a) Schematic of Multi-Layered Elastic System

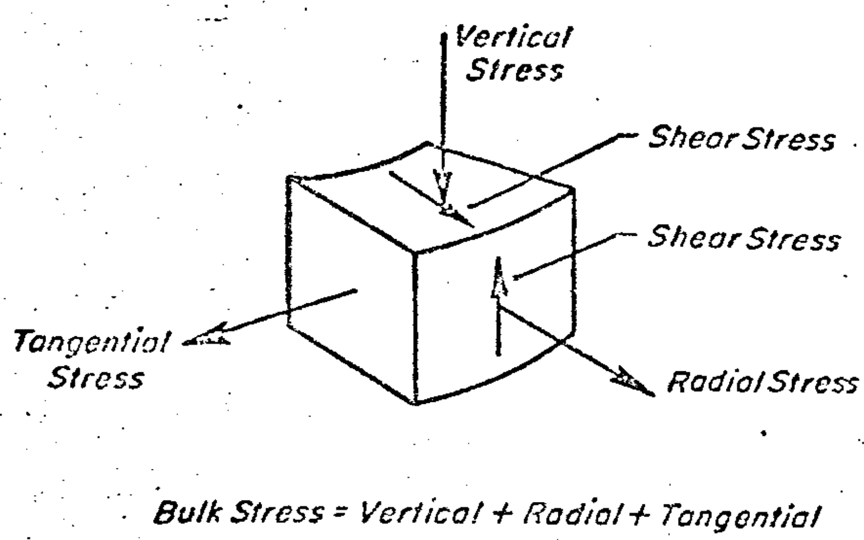
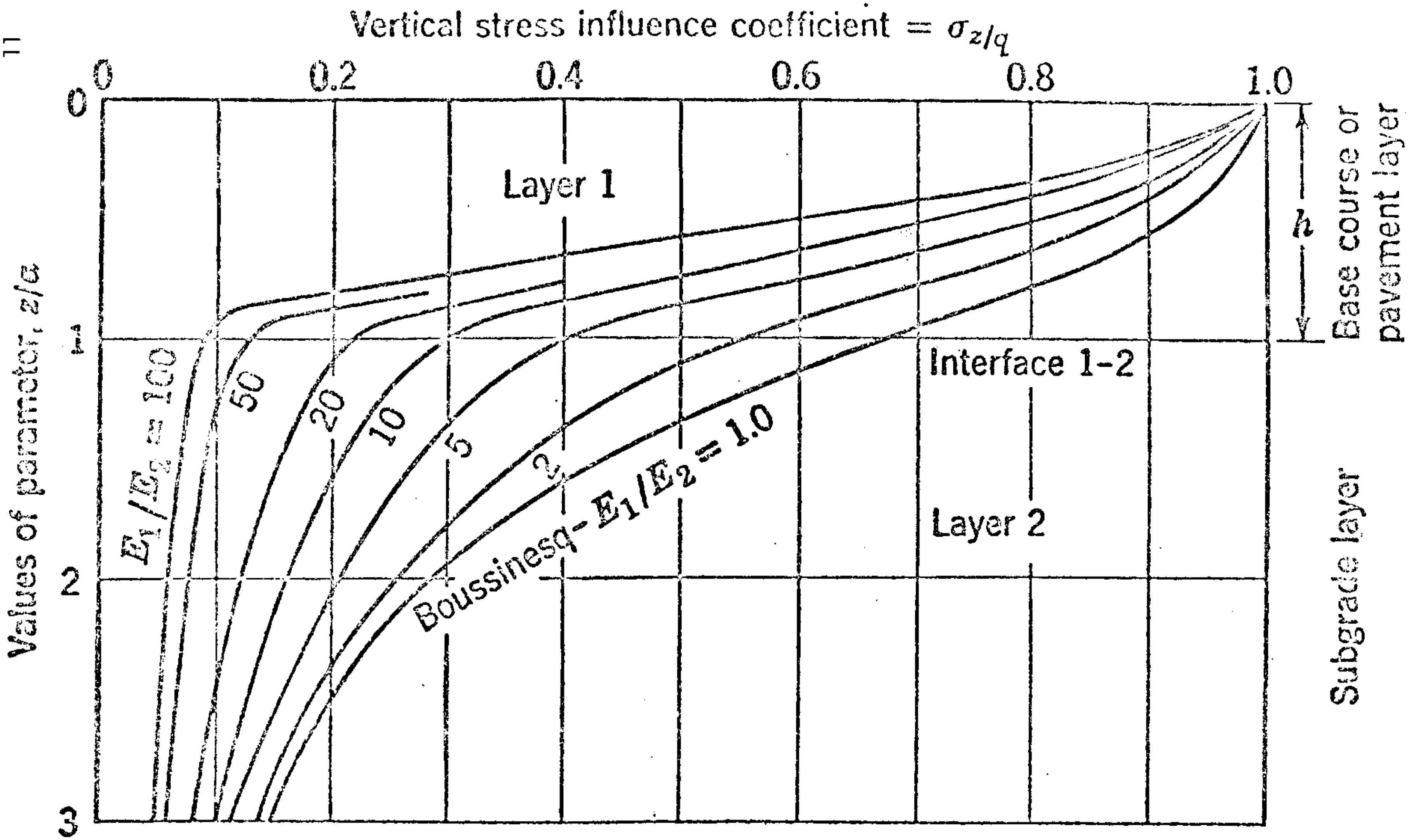


Figure 1(b) Stresses on a Typical Element r-inches from The Axis of the Loaded Area and z-inches Below the Surface, Showing the Nomenclature Used



Example of distribution of vertical stress as a function of depth in a two-layered elastic system, for a = h.

(after Yoder)

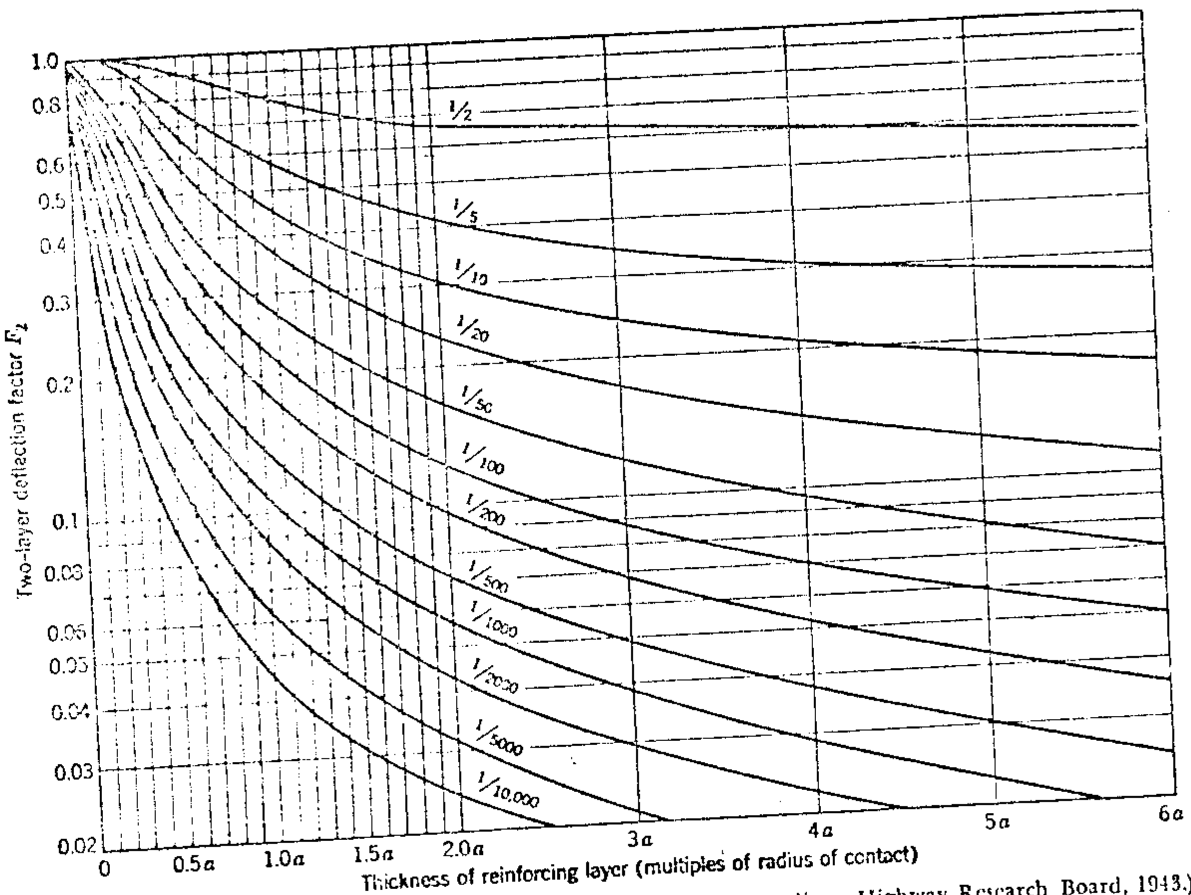


Figure 2.7. Influence values—two-layer theory. (From Burmister, *Proceedings*, Highway Research Board, 1943.)

COMPUTER APPLICATIONS FOR LAYERED PAVEMENT SYSTEMS*

This chapter presents a detailed discussion of five computer solutions to layered systems. All programs have the capability of solving for stresses, strains and displacements for n-layer systems. The limitations of each program are also included.

MULTI-LAYERED ELASTIC SYSTEM

Description

The Multi-Layered Elastic System computer program (CHEV5L) will determine the various component stresses and strains in a three dimensional ideal elastic layered system with a single vertical uniform circular load at the surface of the system Fig. 1(a). The bottom layer of the system is semi-infinite with all other layers of uniform thickness. All layers extend infinitely in the horizontal direction. The top surface of the system is free of shear and all interfaces between layers have full continuity of stresses and displacements.

With a vertical uniform circular load, the system is axisymmetric with the Z axis perpendicular to the layers and extending through the center of the load. Using cylindrical coordinates, any point in the system may be described by an R and Z. R is the horizontal radial distance out from the center of the load and Z is the depth of the point measured vertically from the surface of the system.

The load is described by the total vertical load in pounds and the contact pressure in psi. The load radius is computed by the program. Each layer of the system is described by modulus of elasticity, Poisson's ratio, and thickness in inches. Each layer is numbered with the top layer as 1 and numbering each layer consecutively downward.

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* This section was taken largely from notes prepared by R.G. Hicks, Oregon State University, for the U.S. Forest Service.

Program Operating Notes

The program operates with the various given R and Z values as follows: For every R value a complete set of characterizing functions is developed for all layers, then the stresses and strains are computed at those points represented by that R and each of the given Z values. The stresses calculated are shown in Fig. 1(b). The program then steps to the next R value and computes the stresses and strains at those points represented by each of the given Z values and continues until all combinations of R and Z values are used.

When a given Z value is exactly at an interface between two layers, the program will first compute the stresses and strains at this point using the functions for the upper of the two layers, then will recompute the stresses and strains at this same point using the functions from the lower of the two layers. In the output of the program, a negative Z value indicates that the stresses and strains have been computed at an interface and that the characteristics of the upper layer have been used.

Limitations

The following are limitations of the program and/or method.

1. Number of layers in the system: minimum of two and a maximum of five.
2. Number of points in the system where stresses and strains are to be determined: minimum of one (one R and one Z) to a maximum of 121 (maximum of eleven R and eleven Z).
3. All data are positive, no negative values.
4. Poisson's ratio must <u>not</u> have a value of one.
5. Nonlinear behavior of granular bases and subgrade soils cannot be accounted for.

6. Multiple gears cannot be handled directly. Calculations of critical stresses and strains under multiple gears must be done using the principle of superposition (by hand).

MULTI-LAYERED ELASTIC SYSTEM - ITERATIVE METHOD

Description

The Multi-Layered Elastic System computer program (CHEV5L WITH ITERATION) is also used to determine stresses and strains Fig. 1(b) in a three dimensional elastic layered system with a single vertical uniform circular load. The program is an extension of CHEV5L and has the capability of accounting for variations in the modulus of each material with depth.

As with CHEV5L, all layers are assumed to extend indefinitely in the horizontal direction. The top surface is free of shear and all interfaces between layers have full continuity of stresses and displacements. A vertical uniform circular load is applied and stresses, strains and displacements are calculated at any point in the system described by an R & Z. The load is described by the total vertical load in pounds and contact pressure in psi.

Program Operating Notes

The basic difference between CHEV5L and CHEV5L with iteration lies in the method of assigning modulus of elasticity and Poisson's ratio to each layer. Examination of materials characterizations studies indicates that the modulus of most materials is dependent on the level of stress (or temperature).

In general, the modulus of cohesive soils decreases with increasing repeated stress level σ_d Fig. 2, and is relatively unaffected by small changes in confining pressure. In this program, the modulus of the subgrade (bottom layer) is interpolated from the input modulus-deviator stress relationship. For materials that are not stress dependent, a horizontal relationship must be input. The variation of Poisson's ratio with stress level is less clear although Hicks and Finn found that it remained constant or increased slightly with increasing deviation stress. Poisson's ratio appears not to be significantly affected by confining pressure.

For unstabilized granular materials, the modulus is affected more by confining pressure and is affected to a smaller extent by the magnitude of the deviation stress. As shown in Fig. 3, the modulus of granular materials can be approximated by:

$$M_R = k\sigma_3^n$$

or (1)

$$M_R = \bar{k}\theta^{\bar{n}}$$

where k, $\bar{k}$, n, $\bar{n}$ are constants evaluated from repeated load triaxial test results and σ_3 and θ are confining pressure and the sum of principal stresses ($\theta=\sigma_1+2\sigma_3$ in a conventional triaxial test), respectively. Poisson's ratio has been found to remain relatively constant over a range of stress conditions (19).

Results of repeated load triaxial tests on emulsion mixes have indicated that, at early stages of cures, confining pressure most affects the modulus. The behavior of these materials is very similar to that of granular bases Figure 4. As the curing process progresses, the materials tend to behave more like hot-mix asphalt concrete. Their properties are most affected by temperature and rate (or frequency) of loading as shown in Fig. 5. Results indicate that Poisson's ratio may increase with increasing temperature and is affected only slightly by stress level.

Because the modulus of most materials are dependent on the level of stress, an iterative approach (in which the modulus and stress level interaction can be allowed to close on a system having compatible values of each) was developed as follows:

1. The pavement to be analyzed can be represented by a number of layers consistent with the dimensions of the structural section.
2. The modulus value and Poisson's ratio for each of these layers can be estimated with some degree of accuracy based on the known variation of these values with the estimated stress and environmental conditions.
3. The stresses which would occur in this system under the application of the surface load can be calculated using available computer solutions.
4. The pre-existing stress state owing to overburden pressures can be calculated from knowledge of the densities and dimensions of the pavement materials.
5. The resulting stress state can be obtained by superposition of the load-induced and overburden stresses.
6. The modulus which is compatible with the resulting stress state in each layer can be determined from the appropriate modulus-stress relationship for the materials.
7. The modulus of each layer required by the stress state can be compared with the initially assumed value and the process repeated, using the resulting values, until the initial and final modulus values coincide within a specified accuracy.

In CHEV5L with Iteration, an average modulus under an arbitrary set of dual wheels (center to center spacing of 3R) is calculated using the procedure outlined above. Calculations must be made at the locations indicated in Fig. 6.

Once the iteration process has closed, calculations for stresses, strains and displacements in the system are made (for one wheel loading) as in the case of CHEV5L.

Limitations

The following are limitations of the program and/or method:

1. Number of layers in system: 5 must be used.
2. Number of points in system where stresses and strains are to be determined: minimum of 6 (0,1r, 1-1/2r, 2r, 3r, 4r required) to maximum of 11 R values; minimum of 8 (top, middle and of each non-linear layer) to maximum of 11 Z values; all as shown in Fig. 6.
3. All data are positive, no negative values.
4. Poisson's ratio must not have a value of one.
5. Multiple gears cannot be handled directly. Calculations of critical stresses and strains under multiple gears must be done (by hand) using the principle of superposition.

MULTI-LAYERED ELASTIC SYSTEM - MULTIPLE LOAD OPTION (SHELL BISAR)

Description

The BISAR (Bitumen Structures Analysis in Roads) program is a general purpose program for computing stresses, strains and displacements in elastic layered systems subjected to one or more vertical uniform circular loads applied at the surface of the system. Unlike the CHEV5L programs, the surface loads can be combinations of a vertical normal stress and an unidirectional horizontal stress. All layers extend infinitely in the horizontal direction. The top surface of the system is free of shear. All interfaces between layers have an interface friction factor which can vary between zero (full continuity) and one (frictionless slip) between the layers.

Stresses, strains and displacements are calculated in a cylindrical coordinate system for each vertical load. For more than one load, the cylindrical components are transformed to a Cartesian coordinate system and the effect of the multiple load found by summarizing the stresses, strains and displacements of each wheel. Further, the program calculates only those components which are requested (Table 1)*. If all stresses and strains are calculated, the program calculates the principal stresses and strains and their accompanying directions. The principal directions denote the normals of the planes through the point considered, which are free of shear stress (strain). The highest and lowest of the three principal values give the maximum and minimum normal stresses (strains), and the difference between the principal values divided by two, gives the maximum shear stresses (strains).

For a given problem to be solved using the BISAR program, one needs information regarding:

1. The number of layers;
2. Young's modulus and Poisson's ratio of each layer;
3. The thickness of each layer, except for the bottom one;
4. The interface friction at each interface;
5. The number of loads, the vertical and tangential component of each load, and the position of the loads;
6. The stress, strain and displacement components to be calculated;
7. The number of places where calculations are required along with their position (Cartesian coordinates).

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* Any or all of the types of computations may be requested.

Program Operating Notes:

BISAR consists of a main program and 24 subprograms. The main program reads all the input data defining the numerical problem and controls the subsequent steps in the calculation of the requested stresses, strains and displacements. The output is partly controlled by the main program and by subprograms SYSTEM, CALC, and OUTPUT. Subprograms MACON1, CONPNT, INGRAL and MATRIX give output only when error messages are generated.

The main program can consider several multilayered systems in one run (to a maximum of 99). For each multilayer system, the stresses, strains and displacements can be calculated in an arbitrary number of different positions (to a maximum of 99). Computation proceeds by calculating at each point the stresses, strains and displacements due to each load separately and by transforming these to the Cartesian coordinate system. The Cartesian components are added to those of the preceding loads and by the time the last load has been considered, the total stresses, strains and displacements have been calculated. The computed results are printed (separately) for each position requested.

Limitations

The following are limitations of the program and/or method:

1. Number of layers in the system: maximum of ten, although this can be changed with modifications to the program.
2. Number of systems in one run: maximum of 99.
3. Number of points in the system where stresses and strains can be calculated: maximum of 99.
4. All data are positive, no negative values.
5. Non-linear behavior of granular bases and subgrade soils cannot be accounted for.

MULTI-LAYERED SYSTEM-MULTIPLE LOAD OPTION (ELSYM5)*

Description

The Elastic Layered System computer program (ELSYM5) will determine the various component stresses, strains and displacements along with principal values in a three-dimensional ideal elastic layered system, the layered system being loaded with one or more identical uniform circular loads normal to the surface of the system.

The top surface of the system is free of shear. Each layer is of uniform thickness and extends infinitely in the horizontal direction. All elastic layer interfaces are continuous. The bottom elastic layer may be semi-infinite in thickness or may be given a finite thickness, in which case the program assumes the bottom elastic layer is supported by a rigid base. With a rigid base, the interface between the bottom elastic layer and the base has to be made either fully continuous or slippery.

All locations within the system are described by using the rectangular coordinate system (X,Y,Z) with the XY plane at Z = 0 being the top surface of the elastic system where the loads are applied. The positive Z axis extends vertically down from the surface into the system.

The applied loads are described by any two of the three following items: load in pounds, stress in pounds per square inch, radius of loaded area in inches. The program determines the missing value. Each layer of the system is described by modulus of elasticity, Poisson's ratio and thickness. Each layer is numbered with the top layer as one and numbering each layer consecutively downward.

* This write-up is from text written by Gale Ahlborn, ITTE, University of California at Berkeley, 1972.

Program Operating Notes

The program tests all input data. If any input data is out of range as specified under "Limitations," the problem is terminated for that system with an error message and the program goes on to the next system for operation.

The program uses the convention that compressive stresses are negative and tensile stresses are positive.

The output of the program gives for each depth (Z) all the results for all the XY points. The results for each point are the total results for that point obtained by summing the contribution by each load. When a Z value is determined to be on an interface, the results are determined using the characteristics of the upper of the two layers.

Limitations

Following are the limitations of the program and/or method.

1. Number of different systems for solution: minimum of one, maximum of five.
2. Number of elastic layers in the system: minimum of one, maximum of five.
3. Number of identical uniform circular loads: minimum of one, maximum of ten.*
4. Number of points in the system where results are desired: minimum of one (one XY and one Z), maximum of 100 (ten XY and ten Z).
5. Where there is a rigid base specified, the maximum Z value cannot exceed the depth to the rigid base.
6. All input values except XY positions must be positive.

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* Maximum of five on CDC 3100.

7. Poisson's ratio must <u>not</u> have a value of one. Poisson's ratio for a bottom elastic layer on a rigid base must <u>not</u> be within the range of 0.748 to 0.752.

8. The program uses a truncated series for the integration process that leads to some approximation for the results at and near the surface and at points out at some distance from the load.

MULTI-LAYER ELASTIC THEORY ITERATIVE METHOD-DUAL WHEEL OPTION (PSAD2A)*

Description

PSAD2A is essentially the same as CHEV5L w/iteration except that the former has the added capability of printing stresses, strains, and displacements due to dual wheel configurations. This feature of PSAD2A is not an option; it is performed automatically.

Program Operating Notes

In the case of dual wheels, PSAD2A allows the distance between loads (from edge to edge) to vary between zero and two load radii for the calculation of an average modulus when iterating, whereas CHEV5L w/iteration fixes this distance at one load radii. This can be inferred from the operating notes on CHEV5L w/iteration, where it is stated that an average modulus is calculated under an arbitrary set of dual wheels spaced three radii center to center.

Other than this one difference, the solution method used in PSAD2A is the same as that used by CHEV5L w/iteration.

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* Write-up is based on an excerpt from Report TE 70-5,ITTE, University of California at Berkeley, 1973.

Limitations

1. The number of points for the resilient modulus vs. deviator stress relationship must be at a minimum two and at a maximum twenty. A number outside this range will result in an error message and termination of the run.
2. The resilient modulus-deviator stress curve may be constant or have negative slope, but the first point must have an abcissa (i.e. stress value) of zero. This may require backward extrapolation of experimental data.
3. A maximum of 5 layers is allowed for each system.
4. Consult appropriate limitations for CHEV5L w/iteration.

DISCUSSION

EXAMPLE PROBLEM

To check the operational characteristics of the various programs, an example problem, Fig. 7, was solved, and the output of the various programs compared. The problem (a 3-layered elastic system) represents essentially a full depth asphalt layer over a weak subgrade. Each layer is characterized by a modulus (E_i) and Poisson's ratio (v_i). The load is a 9000 lb. wheel load uniformly distributed with a contact pressure of 80 psi. It should be noted that the contact pressure does not necessarily equal the tire pressure.*

Input formats for each of the programs are discussed in detail in Appendix A. However, Fig. 8 illustrates the data input for the example problem for each of the five programs.

Note, the basic input consists of:

1) Wheel load data;
2) Material properties of each layer; and
3) Thickness of each layer.

Also input are the locations at which calculations are requested. In this particular problem, solutions for stresses and deformations are requested at the surface and at the interfaces of each layer. Calculations are called for at the center (0) and the edge of the loaded area (5.98 inch).

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* If the effect of tire wall is ignored, the contact pressure between the tire and pavement must be equal to the tire pressure. For low pressure tires, contact pressures under the tire are greater near the center. For high pressure tires the reverse is true.

COMPARISON OF RESULTS

For the example problem presented in Fig. 7, the calculated stresses, strains, and deformations are nearly identical. Figure 11a-d is a sample of the output of each program. Items which should be noted are as follows:

1. Stresses. CHEV5L, CHEV5L w/iteration, and PSAD2A output vertical, tangential, radial, shear, and bulk stresses for each position expressed in terms of cylindrical coordinates. Each of these stresses is identified in Fig. 1(b). ELSYM5 calculates normal, shear, principal, and principal shear stresses, presenting them for each position expressed by Cartesian coordinates. Shell BISAR has the capability of computing the stresses shown in Table 1, and outputs each for all positions shown as Cartesian points along with a summary of total stresses.
2. Strains. Radial, tangential, and vertical strains are calculated by PSAD2A and the CHEV5L programs; ELSYM5 outputs the same components of strain as it does of shear for each position requested; and BISAR computes and outputs these strains requested from the list in Table 1, besides giving a summary of total strains.
3. Displacements. ELSYM5 and BISAR each compute three displacements, but present them in a slightly different manner. ELSYM5 displays displacements in a three-dimensional Cartesian coordinate system context, while BISAR displays them not only under the heading of horizontal, vertical, and normal displacements, but also as radial, tangential, and vertical displacements. PSAD2A, CHEV5L, and CHEV5L w/iteration show only vertical deflection.
4. Wheel Load Considerations. The output shown in Fig. 9 is for one uniformly applied vertical load. The load and contact pressure are

input data. These data together with the calculated load radius are printed in the output. All multiple wheel considerations in the Chevron programs must be handled by superposition (see next section). The other three programs, ELSYM5,BISAR, and PSAD2A permit the use of two or more wheels.

5. Sign Notation. A negative sign implies compression when indicated as a stress or strain. Positive, of course, would imply tension. A negative sign for depth implies that the stresses are calculated in the upper layer while a positive sign means the calculation is in the lower layer. This latter notation occurs only at a layer interface.

The data given in Figure 9 were also used to develop Table 2 for comparing results from the various programs. Note that all critical values are essentially the same: both signs and magnitudes agree.

An attempt to compare computational efficiency was made by including CPU time for each run. CPU time refers to central processing unit time, or the seconds during which the computer was actually working on the problem. On the basis of this, a ranking of efficiency is ELSYM5, BISAR, CHEV5L, CHEV5L w/iteration and PSAD2A, though one must consider that CHEV5L and CHEV5L w/iteration were run on a different machine than the others.

MULTIPLE GEAR CONSIDERATIONS

For the Chevron programs (CHEV5L and CHEV5L w/Iteration), output is given for only one circular load. Unfortunately, most vehicles for which pavements are to be designed have either dual or dual-tandem wheel configurations, Figure 10. The question arises, therefore, of how does one determine the critical stresses or deformations for realistic wheel load configurations?

The answer, use of the principle of superposition. This can best be illustrated through an example using the output information presented in Fig. 9(a).

Let us consider the dual wheel configuration. The principle is also applicable to multiple wheel cases and is described as follows:

1) Deformations. The most important deformation one calculates is surface deformation. To calculate the maximum surface deformation resulting from dual wheels one would add the contribution of a second wheel to that of the first. The surface deflection under the first wheel (R=0,Z=0) equals .060 inch, from Fig. 9(a) while that due to the second wheel (R=18.0,Z=0) displaced 18 inches from the first is .042 inch. The total deflection under the first wheel equals the total of both tires or .102 inch. The surface deflection at the edge of one of the dual tires, Fig. 11, would equal Δ_2, which is the sum of Δ_{21} and Δ_{25}. From Figure 11(a), Δ_{21}(R=6.0",Z=0) = .057", and Δ_{25}(R=12.0",Z=0) = .049". Thus, Δ_2 = 0.106". Surface deflections at other intermediate points would be calculated in a similar fashion to determine the maximum value.

2) Stresses and Strains. These two responses would be handled in a similar fashion. Further, the maximum stress and/or strain usually occurs under one of the dual tires. What is needed is to convert the cylindrical coordinates (radial and tangential) to Cartesian coordinates (longitudinal and transverse). Under the first wheel the radial (σ_{r_1}) and tangential (σ_{t_1}) stresses are identical. Under the second wheel (displaced a distance 3 radii, R=18.0, Z=0.0) the radial and tangential stresses are different.

Figure 11(b) shows how they should be added to give the longitudinal and transverse stress conditions. Under most conditions, only the critical stress (or strain) is of interest and this usually occurs at the bottom of the stabilized layer.

For dual-tandem wheel configuration the basic principles described above can also be used. The principal advantage of the Shell BISAR and ELSYM5 programs is that they can accommodate multiple wheel configurations automatically. PSAD2A considers dual wheels automatically.

POTENTIAL APPLICATIONS

The question often arises as to the purpose for using layered theory in the design and evaluation of pavement systems. Particularly so when the present pavement design procedure used in Region 6 (Modified AASHO) appears to be very sound. However, many decisions were made in its development (because of lack of necessary tools or better information) that invalidate extrapolation of the AASHO design procedure to conditions different from that for which it was developed.

In recent years, considerable emphasis has been placed on the development of a more mechanistic design procedure, one which would allow extrapolation to any set of design conditions. This is particularly important because of ever increasing wheel loads such as those from off highway vehicles. Layered theory analysis uses actual load data and fundamental material properties and can properly account for rapidly changing design conditions. This does not mean, however, that layered theory analyses will be a practical design tool, because input to the design process requires sophisticated materials testing and computational equipment. What it does mean is that layered analysis techniques can provide the necessary tools to understand and account for:

1) The impact of off-highway loads on the performance of the pavement system.
2) An evaluation of realistic layer equivalencies for structural materials heretofore not used.
3) Verification or modification of load equivalencies in the design of pavement systems and in the assignment of maintenance responsibilities in the case of dual ownership.

In specific, the following applications are recommended for each of the five programs described in Chapter Three:

1) CHEV5L. This program is the simplest to operate. It should always be considered first to give a "ball park" solution to a particular problem. The most significant limitation is its inability to handle non-linear material problems. Therefore, its use should probably be limited to full-depth asphalt pavements over subgrade.
2) CHEV5L w/Iteration. This program is slower than CHEV5L but does allow one to account for non-linear material behavior. This program should be used where a considerable amount of untreated aggregate is present (e.g. unsurfaced roads).
3) SHELL BISAR. This program, because of its multiple gear option, would be most useful in the evaluation of off-highway loads. Further, the additional capability of horizontal

stresses and ability to vary friction between layers offer capabilities which none of the others can. However, computational experience of the author with the horizontal stress option indicates that excessive time may be used by the computer to converge to a solution. This option should be used only with extreme caution.

4) ELSYM5. This program, similar to the SHELL BISAR model, was found to be the most efficient in terms of computer time. The amount of testing done was very limited, so no definite conclusions can be made in this regard. Applications for ELSYM5 are similar to those of BISAR since multiple loads can be considered.

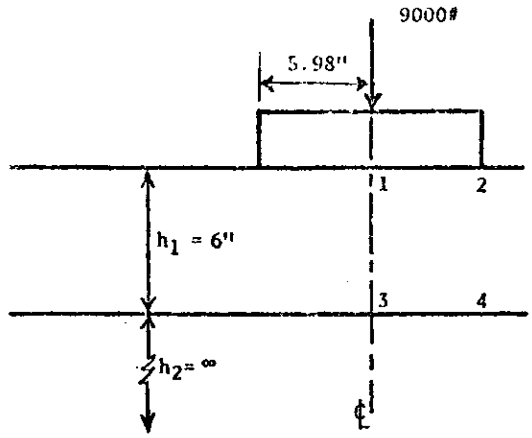
5) PSAD2A. While akin to CHEV5L w/iteration, PSAD2A allows evaluation of stresses, strains and displacements due to a dual wheel load configuration, such as is shown in Figure 11(a). PSAD2A has the capability of handling non-linear material behavior; in this way it is similar to CHEV5L w/iteration.

These five programs (or their equivalent) should provide the U.S. Forest Service with the necessary tools to evaluate almost any unusual design situation.

TABLE 1. STRESSES, STRAINS AND DISPLACEMENTS CALCULATED BY BISAR

Displacements	UR	-	Radial Displacement
	UT	-	Tangential Displacement
	UZ	-	Vertical Displacement
Stresses	SRR	-	Radial Stress
	STT	-	Tangential Stress
	SZZ	-	Vertical Stress
	SRT	-	Radial/Tangential
	SRZ	-	Radial/Vertical
	STZ	-	Tangential/Vertical
Strains	ERR	-	Radial Strain
	ETT	-	Tangential Strain
	EZZ	-	Vertical Strain
	ERT	-	Radial/Tangential
	ERZ	-	Radial/Vertical
	ETZ	-	Tangential/Vertical
Total Displacements	UX	-	x-displacement
	UY	-	y-displacement
Total Stresses	SXX	-	xx component of Total Stress
	SXY	-	xy component of Total Stress
	SXZ	-	xz component of Total Stress
	SYY	-	yy component of Total Stress
	SYZ	-	yz component of Total Stress
Total Strains	EXX	-	xx component of Total Strain
	EXY	-	xy component of Total Strain
	EXZ	-	xz component of Total Strain
	EYY	-	yy component of Total Strain
	EYZ	-	yz component of Total Strain

TABLE 2. COMPARISON OF RESULTS FROM DIFFERENT PROGRAMS



POSITION	ELSYM5	PSAD2A	CHEV5L	CHEV5L w/Iter.	BISAR
VERTICAL STRESS (PSI)					
1	-80.0	-80.0	-80.0	-80.0	-80.1
2	-41.3	-40.0	-41.3	-41.3	-40.1
3	- 6.8	- 6.8	- 6.8	- 6.8	- 6.7
4	- 5.5	- 5.5	- 5.5	- 5.5	- 5.5
VERTICAL STRAIN (in/in x 10^{-4})					
1	4.9	4.9			4.9
2	3.9	3.9			3.9
3	- 7.0	- 7.0			- 7.0
4	- 4.9	- 4.8			- 4.8
TANGENTIAL STRESS (PSI)					
1	-284.7	-284.7	-284.7	-284.7	-284.0
2	-212.1	-211.0	-212.1	-212.1	-210.3
3	254.9	254.9	254.9	254.9	254.2
4	191.6	191.6	191.6	191.6	190.8
TANGENTIAL STRAIN (in/in x 10^{-4})					
1	- 4.6	- 4.6	- 4.6	- 4.6	- 4.6
2	- 4.1	- 4.1	- 4.1	- 4.1	- 4.1
3	5.2	5.2	5.2	5.2	5.2
4	4.3	4.3	4.3	4.3	4.3
RADIAL STRESS (PSI)					
1	-284.7	-284.7	-284.7	-284.7	-284.0
2	-183.5	-182.2	-183.5	-183.5	-181.3
3	254.9	254.9	254.9	254.9	254.2
4	158.5	158.3	158.5	158.5	157.6
RADIAL STRAIN (in/in x 10^{-4})					
1	- 4.6	- 4.6	- 4.6	- 4.6	- 4.6
2	- 2.7	- 2.7	- 2.7	- 2.7	- 2.7
3	5.2	5.2	5.2	5.2	5.2
4	2.8	2.8	2.8	2.8	2.8
CPU TIME (SEC.)					
	1.731*	17.743*	11.036**	UA+	2.606*

* Run on CDC CYBER 73/74
** Run on UNIVAC 1108
\+ Unavailable

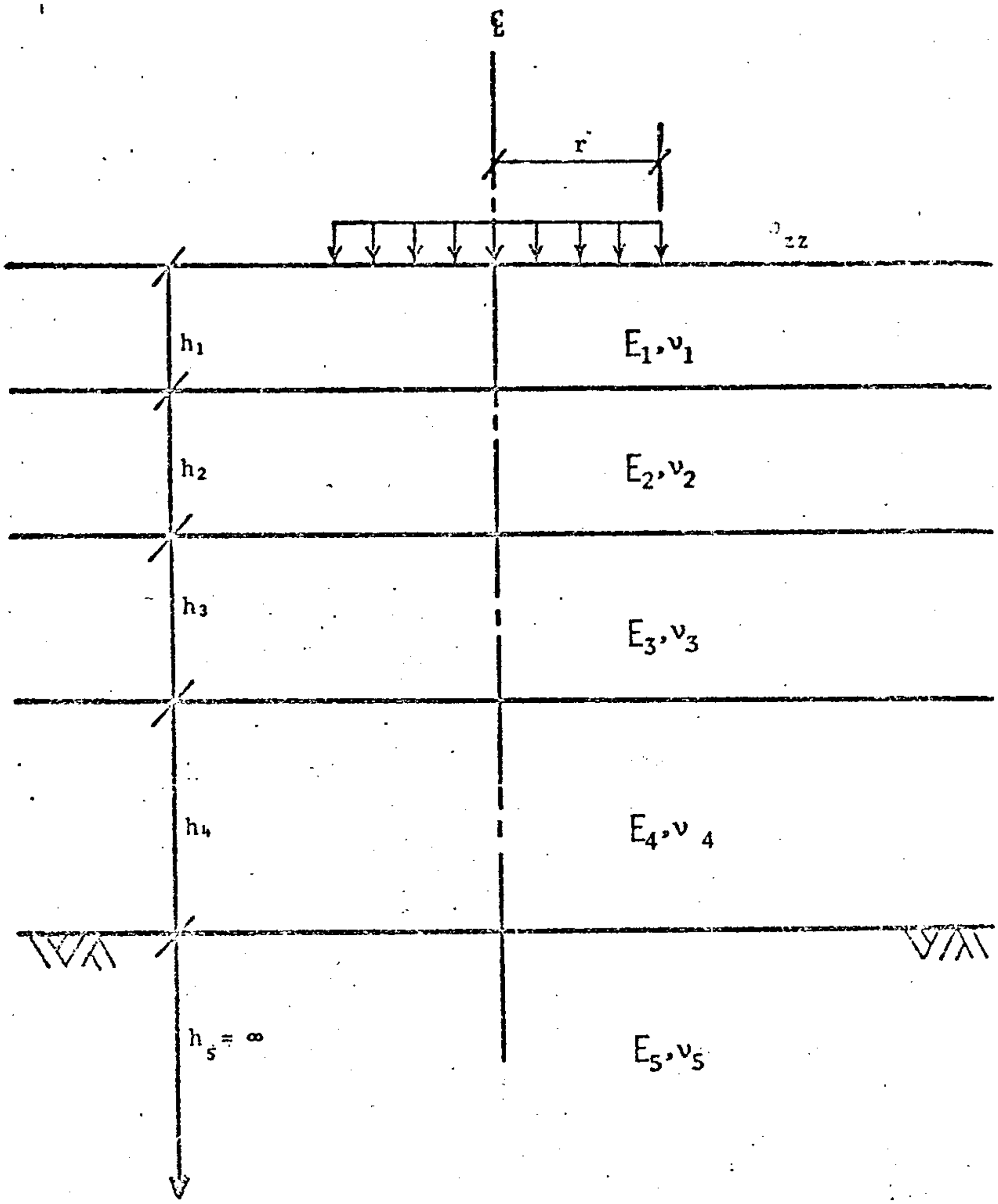


Fig. 1(a) Schematic of Multi-Layered Elastic System

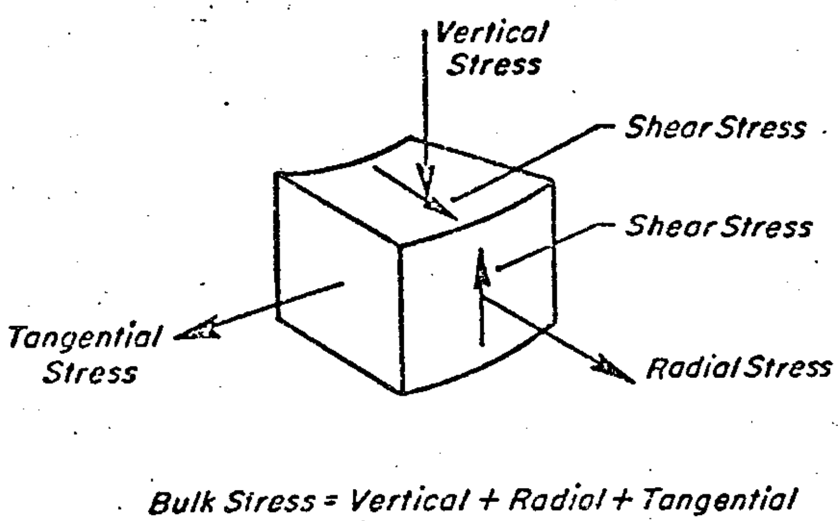


Figure 1(b) Stresses on a Typical Element r-inches from The Axis of the Loaded Area and z-inches Below the Surface, Showing the Nomenclature Used

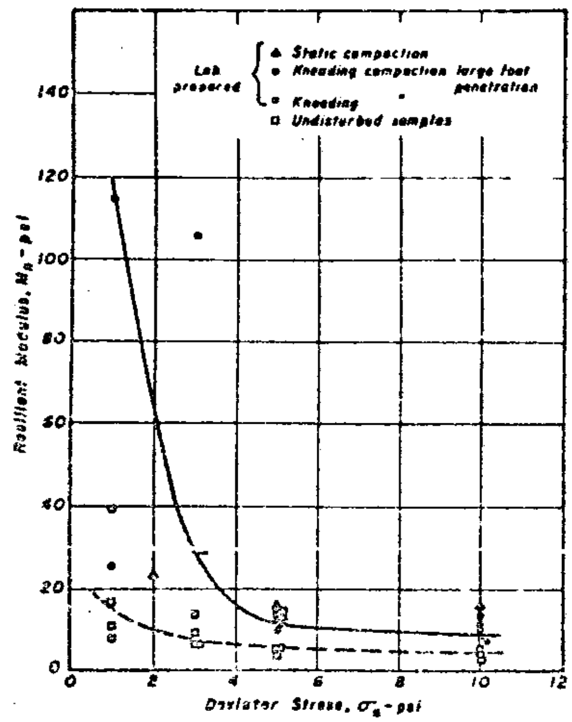


Figure 2 Results of Repeated Load Tests, Morro Bay Subgrade Material

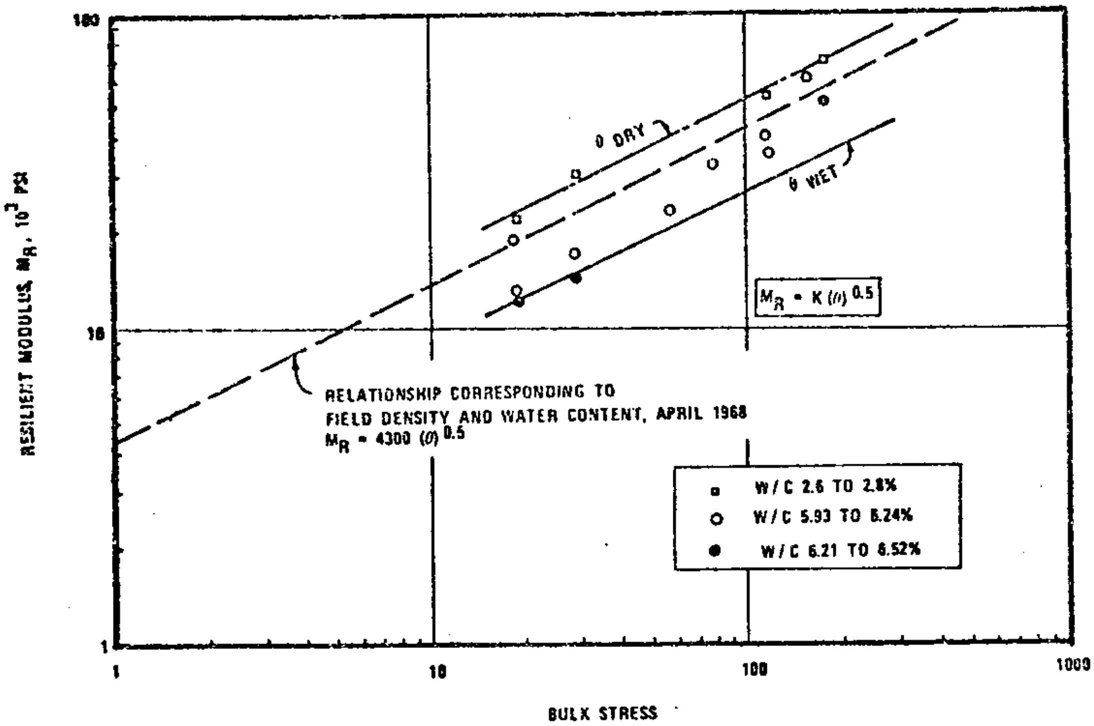


FIGURE 3 RESILIENT MODULUS (M_R) AS A FUNCTION OF BULK STRESS ($\theta = \sigma_1 + 2\sigma_3$)
SECTION 1 AGGREGATE BASE

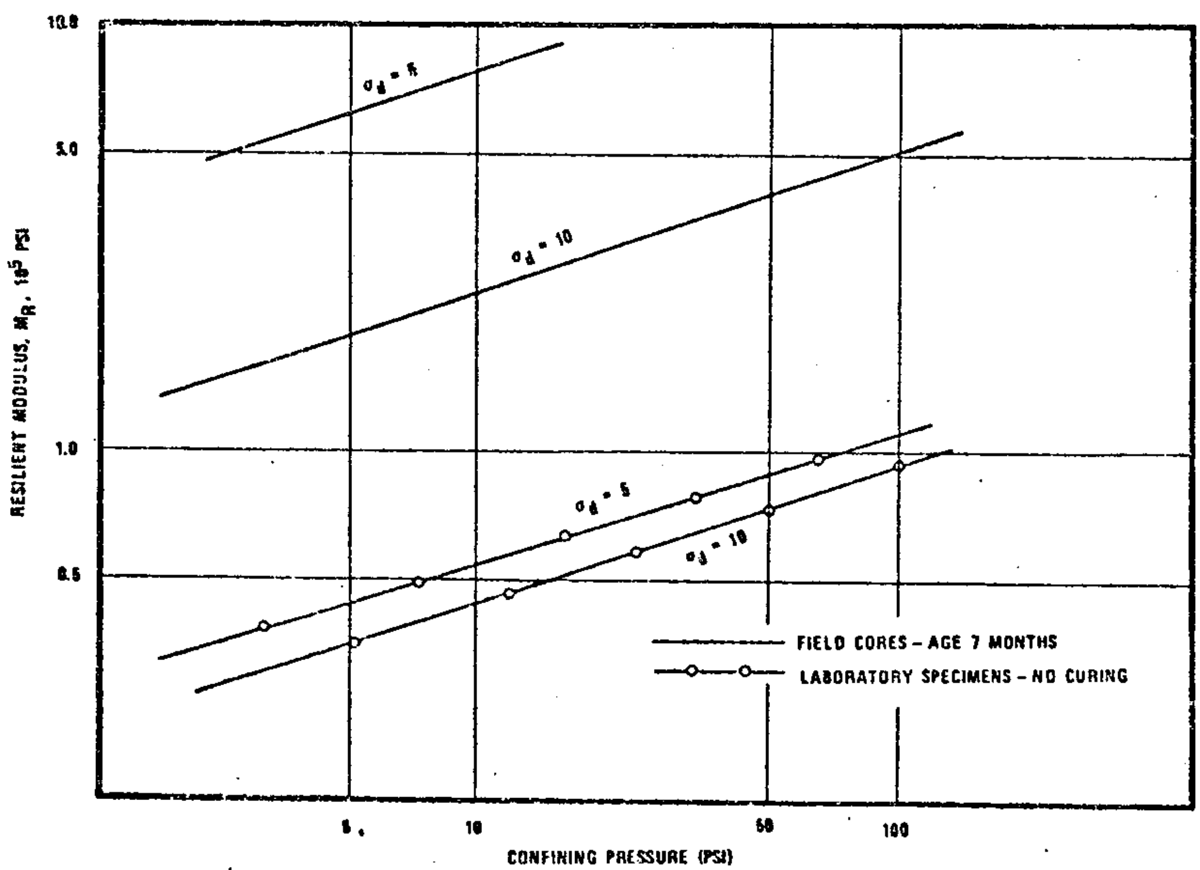


FIGURE 4 COMPARISON OF RESILIENT MODULUS vs. CONFINING PRESSURE FOR SPECIMENS OF EMULSION TREATED SPECIAL AGGREGATE AT DIFFERENT STAGES OF CURING

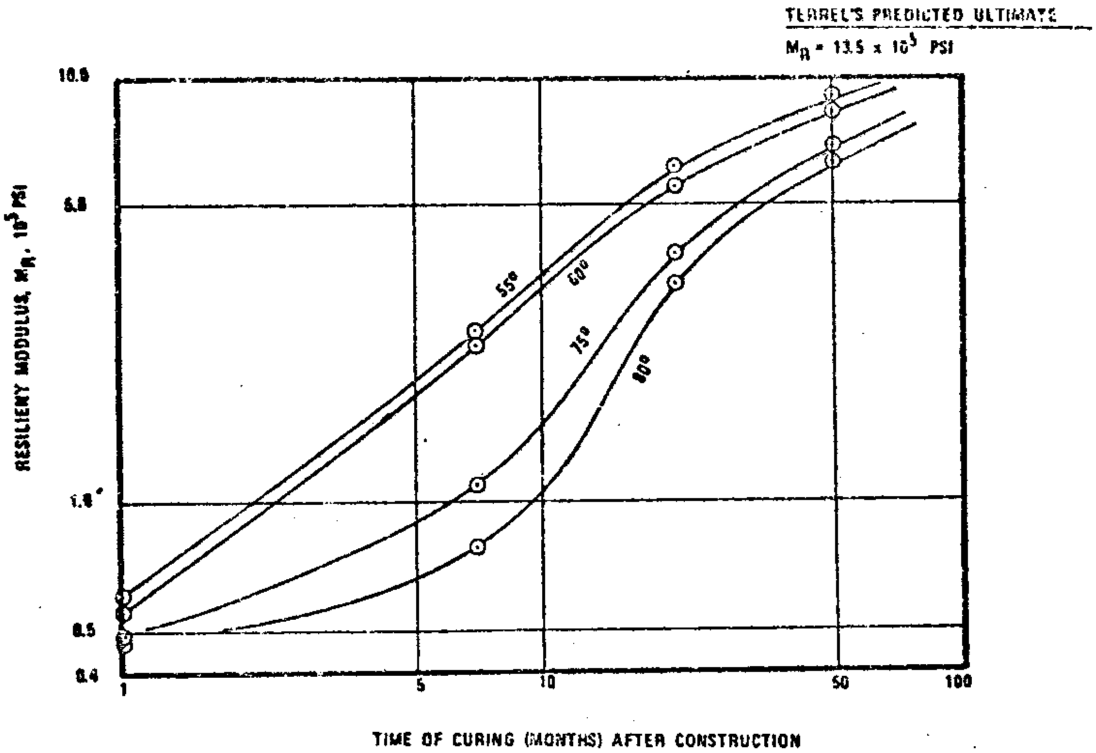


Figure 5(a) Risilient Modulus Vs. Curing Time for Emulsion Treated Aggregate

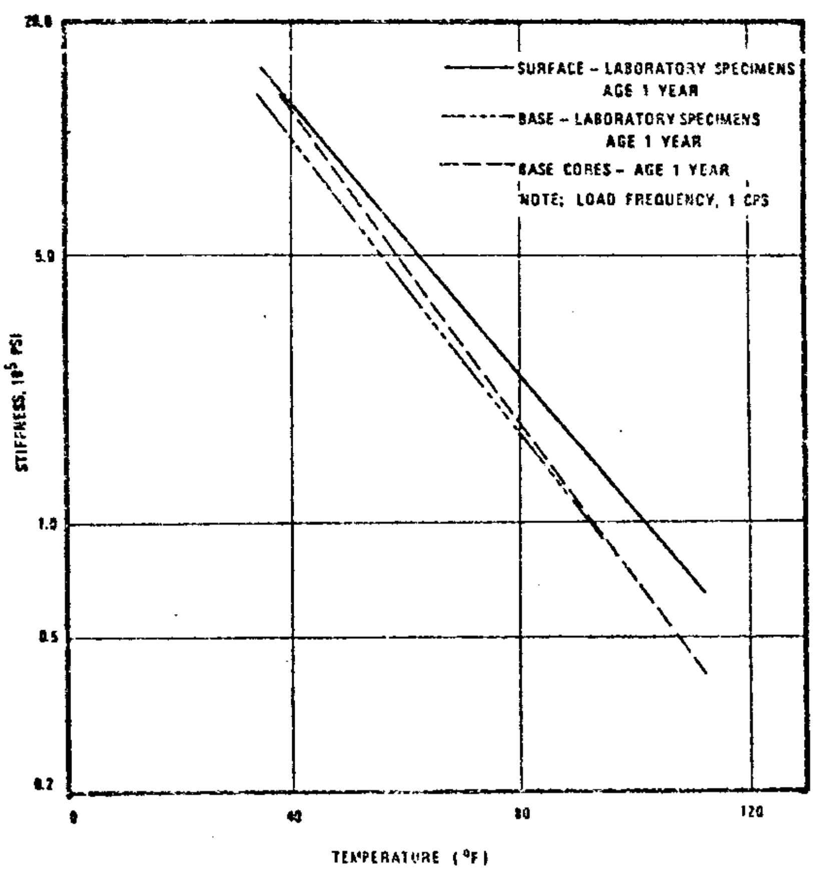
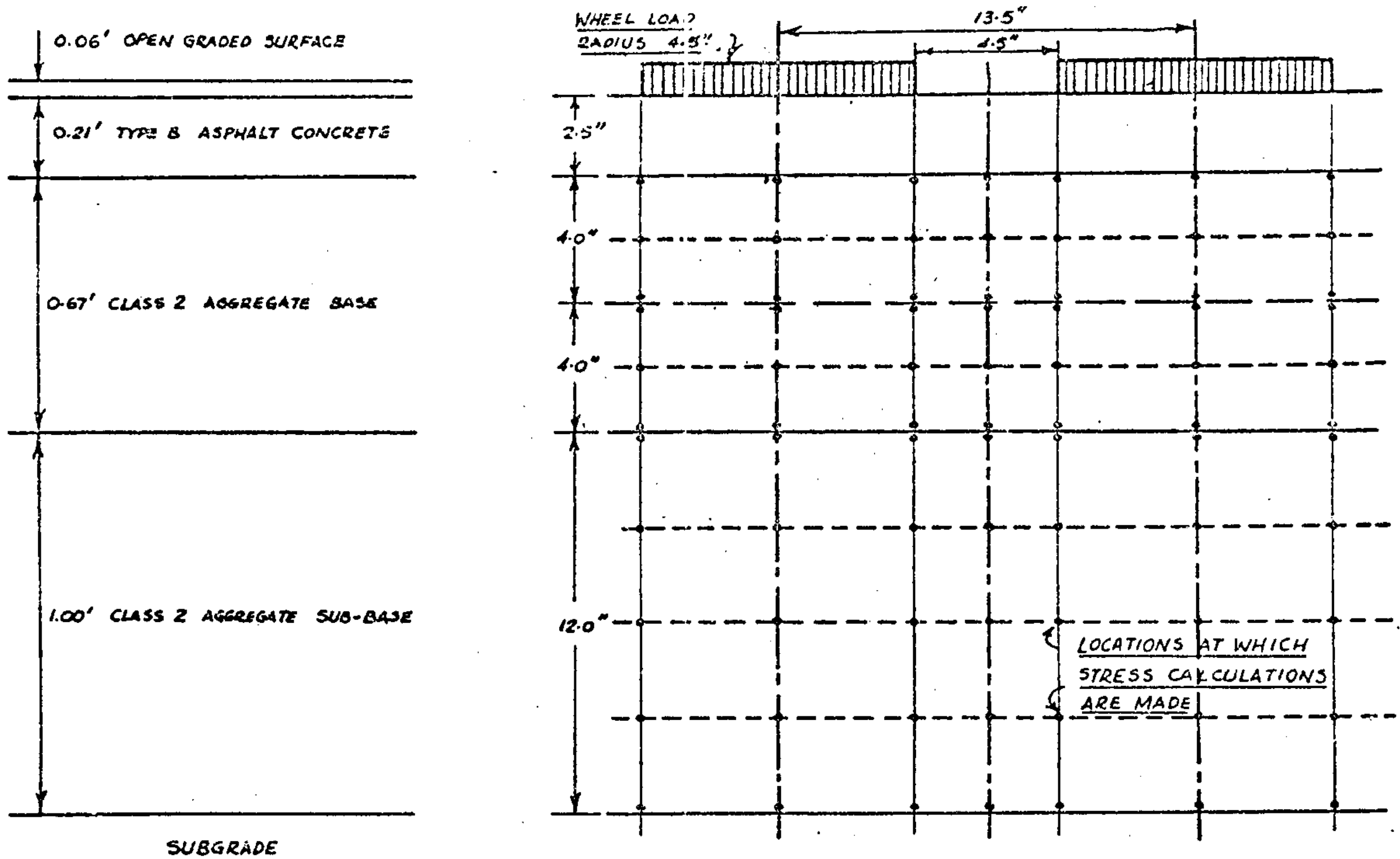


Figure 5(b) Complex Modulus Vs. Teperature for Asphalt Concrete, Surface and Base



(a) Structural Section

(b) Computer Representation

Figure 6 Morro Bay Pavement, (V-SLO-56-C,D)

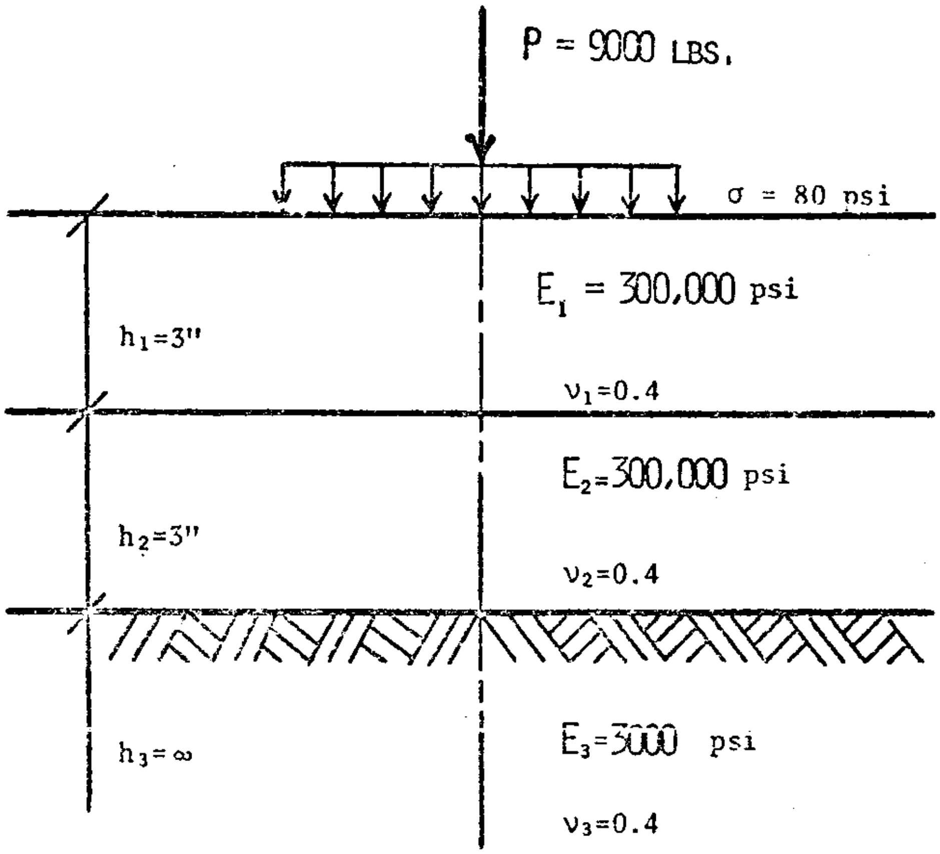


Fig. 7 Example Problem - Layered Analysis

```
1   TEST RUN 3 LAYER SYSTEM R.G. HICKS
2   9000.   30.0
3   3 30000.   0.4 30000.   0.4   3000.   0.4
4   3.0   3.0
5   2   0.0   5.98
6   3   0.0   3.0   6.0
```

(a) CHEV 5L

```
1   1
2   TEST RUN 3 LAYER SYSTEM R.G. HICKS
3   9000.   30.   1
4   3 30000.   0.4 30000.   0.4   3000.   0.4
5   300000.   0.0   300000.   0.0
6   150.0   150.0
7   3   0.0   3000.   10.   3000.   0.0   3000
8   3.0   3.0
9   2   2   0.0   5.98
10  3   0.0   3.0   6.0
```

(b) CHEV 5L With Iteration

```
1   TEST RUN 3 LAYER SYSTEM R.G. HICKS   8 20 1975
2   1
3   3   1
4   3.0E05   4.0E-01   3.0E00   3.0E05
5   3.0E05   4.0E-01   3.0E00   3.0E05
6   3.0E03   4.0E-01
7   1
8   STRS   8.0E01   5.984E00   0.0E00   0.0E00   0.0E00   0.0E00
9   UR UT UZ SRR STT SZZ   ERR ETT EZZ
10  8
11  1   0.0E00   0.00E00   0.0E00   0.0E00
12  1   0.0E00   5.98E00   0.0E00   0.0E00
13  1   0.0E00   0.0E00   3.0E00   0.0E00
14  2   0.0E00   0.0E00   3.0E00   0.0E00
15  2   0.0E00   0.0E00   6.0E00   0.0E00
16  2   0.0E00   5.98E00   6.0E00   0.0E00
17  3   0.0E00   0.0E00   6.0E00   0.0E00
18  3   0.0E00   5.98E00   6.0E00   0.0E00
```

(c) Shell Bisar

Figure 8 Input for Example Problem

1	1							
2	999 TRIAL RUN ELSYM5 10-28-75							
3	3	1	2	3				
4	1	3.0	0.4	300000.				
5	2	3.0	0.4	300000.				
6	3		0.4	3000.				
7		9000.		80.				
8		0.		0.				
9		0.		0.				
10		0.		5.98				
11	0.	3.	6.					

(d) ELSYM5

1	1							
2	TRIAL RUN PSAD2A 10-28-75							
3	9000.	80.			1.		3	
4	300000.	0.4		300000.	0.	150.		1.5
5	300000.	0.4		300000.	0.	150.		1.5
6	300000.	0.4		300000.	0.	150.		1.5
7	300000.	0.4		300000.	0.	150.		1.5
8	3000.	0.4						
9	0.	3000.		10.	3000.	20.		3000.

(e) PSAD2A

Figure 8 (cont.)

THE PROBLEM PARAMETERS ARE

TOTAL LOAD.. 9000.00 LBS

TIRE PRESSURE.. 80.00 PSI

LOAD RADIUS.. 5.98 IN.

LAYER 1 HAS MODULUS 300000 POISSONS RATIO .400 AND THICKNESS 1.5 IN.
LAYER 2 HAS MODULUS 300000 POISSONS RATIO .400 AND THICKNESS 1.5 IN.
LAYER 3 HAS MODULUS 300000 POISSONS RATIO .400 AND THICKNESS 1.5 IN.
LAYER 4 HAS MODULUS 300000 POISSONS RATIO .400 AND THICKNESS 1.5 IN.
LAYER 5 HAS MODULUS 3000 POISSONS RATIO .400 AND IS SEMI-INFINITE.

		STRESSES					DISPLACEMENT	STRAINS		
R	Z	VERTICAL	TANGENTIAL	RADIAL	SHEAR	BULK	VERTICAL	RADIAL	TANGENTIAL	SHEAR
0	0	-8.000E 01	-2.847E 02	-2.847E 02	0	-6.495E 02	6.032E-02	-4.628E-04	-4.628E-04	0SLC
0	-1.5	-7.014E 01	-1.398E 02	-1.398E 02	0	-3.496E 02	6.078E-02	-1.861E-04	-1.861E-04	0
0	1.5	-7.014E 01	-1.398E 02	-1.398E 02	0	-3.498E 02	6.078E-02	-1.861E-04	-1.861E-04	0
0	-3.0	-4.617E 01	-1.158E 01	-1.158E 01	0	-6.934E 01	6.078E-02	3.840E-05	3.840E-05	0
0	3.0	-4.617E 01	-1.158E 01	-1.158E 01	0	-6.934E 01	6.078E-02	3.840E-05	3.840E-05	0
0	-4.5	-1.906E 01	1.134E 02	1.134E 02	0	2.078E 02	6.042E-02	2.542E-04	2.542E-04	0
0	4.5	-1.906E 01	1.134E 02	1.134E 02	0	2.078E 02	6.042E-02	2.542E-04	2.542E-04	0
0	-6.0	-6.766E 00	2.549E 02	2.549E 02	0	5.031E 02	5.963E-02	5.189E-04	5.189E-04	0
0	6.0	-6.766E 00	-1.917E 00	-1.917E 00	0	-1.060E 01	5.963E-02	5.189E-04	5.189E-04	0
6.0	0	-4.126E 01	-2.121E 02	-1.835E 02	-1.130E-09	-4.369E 02	5.653E-02	-2.760E-04	-4.073E-04	-2.435E-03SLO
6.0	-1.5	-3.308E 01	-9.738E 01	-9.527E 01	-4.517E 01	-2.257E 02	5.692E-02	-1.436E-04	-1.535E-04	-9.177E-04
6.0	1.5	-3.308E 01	-9.738E 01	-9.527E 01	-4.517E 01	-2.257E 02	5.692E-02	-1.436E-04	-1.535E-04	-9.177E-04
6.0	-3.0	-2.162E 01	-3.224E 00	-1.110E 01	-4.770E 01	-3.594E 01	5.699E-02	-3.870E-06	3.288E-05	1.966E-04
6.0	3.0	-2.162E 01	-3.224E 00	-1.110E 01	-4.770E 01	-3.594E 01	5.699E-02	-3.870E-06	3.288E-05	1.966E-04
6.0	-4.5	-1.073E 01	8.919E 01	7.248E 01	-3.472E 01	1.509E 02	5.676E-02	1.370E-04	2.150E-04	1.286E-03
6.0	4.5	-1.073E 01	8.919E 01	7.248E 01	-3.472E 01	1.509E 02	5.676E-02	1.370E-04	2.150E-04	1.286E-03
6.0	-6.0	-5.525E 00	1.910E 02	1.545E 02	-1.062E 00	3.446E 02	5.622E-02	2.800E-04	4.349E-04	2.501E-03
6.0	6.0	-5.525E 00	-1.731E 00	-2.063E 00	-1.062E 00	-9.319E 00	5.622E-02	2.800E-04	4.349E-04	2.501E-03
10.0	0	8.200E-01	-1.253E 02	-8.118E 01	-8.090E-10	-2.056E 02	5.183E-02	-1.043E-04	-3.107E-04	-3.101E-03SLO
10.0	-1.5	-2.192E-01	-6.189E 01	-4.787E 01	-2.087E 01	-1.100E 02	5.212E-02	-7.676E-05	-1.422E-04	-1.419E-03
10.0	1.5	-2.192E-01	-6.189E 01	-4.787E 01	-2.087E 01	-1.100E 02	5.212E-02	-7.676E-05	-1.422E-04	-1.419E-03
10.0	-3.0	-1.556E 00	-7.471E-01	-7.639E 00	-2.919E 01	-9.945E 00	5.223E-02	-2.239E-05	9.773E-06	9.753E-05
10.0	3.0	-1.556E 00	-7.471E-01	-7.639E 00	-2.919E 01	-9.945E 00	5.223E-02	-2.239E-05	9.773E-06	9.753E-05
10.0	-4.5	-3.294E 00	6.064E 01	3.465E 01	-2.294E 01	9.200E 01	5.213E-02	3.905E-05	1.603E-04	1.600E-03
10.0	4.5	-3.294E 00	6.064E 01	3.465E 01	-2.294E 01	9.200E 01	5.213E-02	3.905E-05	1.603E-04	1.600E-03
10.0	-6.0	-4.179E 00	1.252E 02	7.244E 01	-1.087E 00	1.935E 02	5.182E-02	8.008E-05	3.264E-04	3.257E-03
10.0	6.0	-4.179E 00	-1.546E 00	-2.034E 00	-1.087E 00	-7.719E 00	5.182E-02	8.008E-05	3.264E-04	3.257E-03
12.0	0	2.905E-01	-1.049E 02	-5.800E 01	1.671E-09	-1.625E 02	4.938E-02	-5.401E-05	-2.723E-04	-3.257E-03SLO
12.0	-1.5	-4.137E-01	-5.214E 01	-3.371E 01	-1.649E 01	-8.626E 01	4.963E-02	-4.228E-05	-1.283E-04	-1.535E-03
12.0	1.5	-4.137E-01	-5.214E 01	-3.371E 01	-1.649E 01	-8.626E 01	4.963E-02	-4.228E-05	-1.283E-04	-1.535E-03
12.0	-3.0	-1.566E 00	-1.264E 00	-5.478E 00	-2.248E 01	-8.308E 00	4.972E-02	-1.449E-05	5.178E-06	6.193E-05
12.0	3.0	-1.566E 00	-1.264E 00	-5.478E 00	-2.248E 01	-8.308E 00	4.972E-02	-1.449E-05	5.178E-06	6.193E-05
12.0	-4.5	-2.970E 00	4.964E 01	2.326E 01	-1.743E 01	6.997E 01	4.964E-02	1.540E-05	1.382E-04	1.553E-03
12.0	4.5	-2.970E 00	4.964E 01	2.326E 01	-1.743E 01	6.997E 01	4.964E-02	1.540E-05	1.382E-04	1.553E-03
12.0	-6.0	-3.651E 00	1.023E 02	4.849E 01	-1.024E 00	1.471E 02	4.940E-02	3.010E-05	2.812E-04	3.363E-03
12.0	6.0	-3.651E 00	-1.347E 00	-1.925E 00	-1.024E 00	-6.963E 00	4.940E-02	3.010E-05	2.812E-04	3.363E-03
17.9	0	4.105E-01	-6.043E 01	-1.425E 01	2.937E-10	-7.427E 01	4.225E-02	3.253E-05	-1.830E-04	-3.283E-03SLO
17.9	-1.5	-3.765E-01	-3.096E 01	-9.007E 00	-8.335E 00	-4.115E 01	4.236E-02	9.094E-06	-8.963E-05	-1.606E-03SLO
17.9	1.5	-3.765E-01	-3.096E 01	-9.007E 00	-8.335E 00	-4.115E 01	4.236E-02	9.094E-06	-8.963E-05	-1.606E-03SLO
17.9	-3.0	-1.224E 00	-1.727E 00	-3.273E 00	-1.126E 01	-6.234E 00	4.240E-02	-6.962E-06	2.076E-07	3.725E-06
17.9	3.0	-1.224E 00	-1.727E 00	-3.273E 00	-1.126E 01	-6.234E 00	4.240E-02	-6.962E-06	2.076E-07	3.725E-06
17.9	-4.5	-2.059E 00	2.745E 01	3.275E 00	-8.739E 00	2.867E 01	4.237E-02	-2.294E-05	8.989E-05	1.613E-03
17.9	4.5	-2.059E 00	2.745E 01	3.275E 00	-8.739E 00	2.867E 01	4.237E-02	-2.294E-05	8.989E-05	1.613E-03
17.9	-6.0	-2.427E 00	5.711E 01	8.136E 00	-4.233E-01	6.282E 01	4.226E-02	-4.579E-05	1.828E-04	3.279E-03
17.9	6.0	-2.427E 00	-1.030E 00	-1.520E 00	-4.233E-01	-4.977E 00	4.226E-02	-4.579E-05	1.828E-04	3.279E-03
23.9	0	1.756E-01	-3.512E 01	5.263E 00	1.388E-09	-2.968E 01	3.576E-02	6.386E-05	-1.246E-04	-2.980E-03SLO
23.9	-1.5	-2.246E-01	-1.841E 01	8.598E-01	-4.320E 00	-1.777E 01	3.591E-02	2.771E-05	-6.220E-05	-1.489E-03SLO
23.9	1.5	-2.246E-01	-1.841E 01	8.598E-01	-4.320E 00	-1.777E 01	3.591E-02	2.771E-05	-6.220E-05	-1.489E-03SLO
23.9	-3.0	-7.941E-01	-1.579E 00	-2.264E 00	-5.076E 00	-4.618E 00	3.593E-02	-4.317E-06	-1.213E-06	-2.901E-05
23.9	3.0	-7.941E-01	-1.579E 00	-2.264E 00	-5.076E 00	-4.618E 00	3.593E-02	-4.317E-06	-1.213E-06	-2.901E-05
23.9	-4.5	-1.337E 00	1.523E 01	-5.344E 00	-4.639E 00	8.552E 00	3.582E-02	-3.634E-05	5.968E-05	1.428E-03
23.9	4.5	-1.337E 00	1.523E 01	-5.344E 00	-4.639E 00	8.552E 00	3.582E-02	-3.634E-05	5.968E-05	1.428E-03
23.9	-6.0	-1.584E 00	3.214E 01	-9.416E 00	-6.419E-01	2.114E 01	3.577E-02	-7.211E-05	1.218E-04	2.913E-03
23.9	6.0	-1.584E 00	-7.241E-01	-1.140E 00	-6.419E-01	-3.468E 00	3.577E-02	-7.211E-05	1.218E-04	2.913E-03

I FTNO 0060 STOP

Figure 9(a) Output for Example Problem (CHEV5L and CHEV5L with iteration)

```
ELSYM5  3/72 - 3,  ELASTIC LAYERED SYSTEM WITH ONE TO TEN NORMAL IDENTICAL CIRCULAR UNIFORM LOAD(S)

ELASTIC SYSTEM  1 - TRIAL RUN ELSYM5 10-28-75

  Z=    0.00  LAYER NO  1

  X=          0.00          0.00
  Y=          0.00          5.98

NORMAL-STRESSES
 SXX    -.2847E+03 -.2121E+03
 SYY    -.2847E+03 -.1835E+03
 SZZ    -.9000E+02 -.4126E+02

SHEAR STRESSES
 SXY    0.          0.
 SXZ    0.          0.
 SYZ    0.          -.3626E-11

PRINCIPAL STRESSES
 PS 1  -.9000E+02 -.4126E+02
 PS 2  -.2847E+03 -.1835E+03
 PS 3  -.2847E+03 -.2121E+03

PRINCIPAL SHEAR STRESSES
 PSS 1  .1024E+03  .8542E+02
 PSS 2  .1024E+03  .7114E+02
 PSS 3  0.         .1428E+02

DISPLACEMENTS
 UX     0.          0.
 UY     0.          -.2435E-02
 UZ     .6032E-01   .5653E-01

NORMAL STRAINS
 EXX    -.4628E-03 -.4073E-03
 EYY    -.4628E-03 -.2740E-03
 EZZ     .4926E-03  .3900E-03

SHEAR-STRAINS
 EXY    0.          0.
 EYZ    0.          0.
 EXZ    0.          -.3385E-16

PRINCIPAL STRAINS
 PE 1   .4926E-03   .3900E-03
 PE 2  -.4628E-03  -.2740E-03
 PE 3  -.4628E-03  -.4073E-03

PRINCIPAL SHEAR STRAINS
 PSE 1  .9555E-03   .7972E-03
 PSE 2  .9555E-03   .6639E-03
 PSE 3  0.          .1333E-03
```

Figure 9(b)   Output for Example Problem (ELSYM5)

************ TRIAL RUN PSAD2A 10-28-75 **************

THE PROBLEM PARMETERS ARE,

TOTAL LOAD PER WHEEL 9000.00 LBS

TIRE PRESSURE.. 90.00 PSI

LOAD RADIUS.. 5.98 IN.

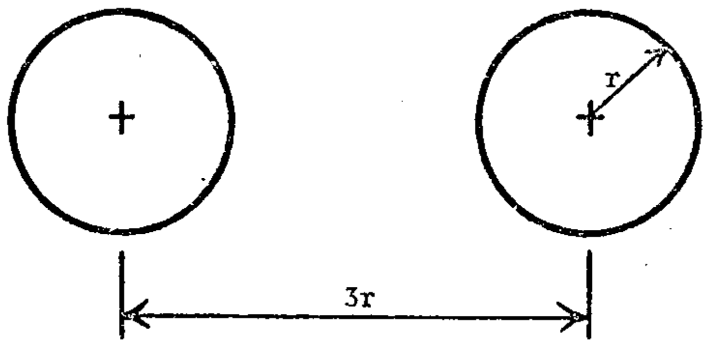
AYER 1	HAS MODULUS	300000.	FROM 300000.0	THETA **	0.000	POISSONS RATIO	.400	THICKNESS 1.5 IN.	UNIT.WT. 150.0 PCF
AYER 2	HAS MODULUS	300000.	FROM 300000.0	THETA **	0.000	POISSONS RATIO	.400	THICKNESS 1.5 IN.	UNIT.WT. 150.0 PCF
AYER 3	HAS MODULUS	300000.	FROM 300000.0	THETA **	0.000	POISSONS RATIO	.400	THICKNESS 1.5 IN.	UNIT.WT. 150.0 PCF
AYER 4	HAS MODULUS	300000.	FROM 300000.0	THETA **	0.000	POISSONS RATIO	.400	THICKNESS 1.5 IN.	UNIT.WT. 150.0 PCF
AYER 5	HAS MODULUS	3000.	FROM DEVIATOR STRESS RELATIONSHIP			POISSONS RATIO	.400	AND IS SEMI-INFINITE	

		STRESSES					DISPLACEMENT	STRAINS		
R	Z	VERTICAL	TANGENTIAL	RADIAL	SHEAR	BULK	VERTICAL	RADIAL	TANGENTIAL	VERTICAL
.00	0.00	-3.000E+01	-2.347E+02	-2.347E+02	0.	-6.495E+02 *	6.032E-02 *	-4.628E-04	-4.628E-04	4.926E-04SLOW
.00	-1.50	-7.014E+01	-1.398E+02	-1.338E+02	0.	-3.498E+02 *	6.073E-02 *	-1.861E-04	-1.861E-04	1.391E-04
.00	1.50	-7.014E+01	-1.398E+02	-1.398E+02	0.	-3.498E+02 *	6.079E-02 *	-1.861E-04	-1.861E-04	1.391E-04
.00	2.25	-5.930E+01	-7.436E+01	-7.436E+01	0.	-2.030E+02 *	6.083E-02 *	-6.966E-05	-6.966E-05	6.329E-07
.00	-3.00	-4.617E+01	-1.159E+01	-1.159E+01	0.	-6.934E+01 *	6.078E-02 *	3.840E-05	3.840E-05	-1.230E-04
.00	3.00	-4.617E+01	-1.159E+01	-1.159E+01	0.	-6.934E+01 *	6.078E-02 *	3.840E-05	3.840E-05	-1.230E-04
.00	3.75	-3.240E+01	5.042E+01	5.042E+01	0.	6.843E+01 *	6.064E-02 *	1.440E-04	1.440E-04	-2.425E-04
.00	-4.50	-1.956E+01	1.138E+02	1.138E+02	0.	2.078E+02 *	6.042E-02 *	2.542E-04	2.542E-04	-3.693E-04
.00	4.50	-1.956E+01	1.138E+02	1.138E+02	0.	2.078E+02 *	6.042E-02 *	2.542E-04	2.542E-04	-3.693E-04
.00	5.25	-1.038E+01	1.811E+02	1.811E+02	0.	3.517E+02 *	6.008E-02 *	3.764E-04	3.764E-04	-5.183E-04
.00	-6.00	-6.766E+00	2.549E+02	2.549E+02	0.	5.031E+02 *	5.963E-02 *	5.139E-04	5.139E-04	-7.024E-04
.00	6.00	-6.766E+00	-1.917E+00	-1.917E+00	0.	-1.060E+01 *	5.363E-02 *	5.139E-04	5.139E-04	-1.744E-03
.93	0.00	-4.000E+01	-2.110E+02	-1.822E+02	4.899E-13	-4.332E+02 *	5.652E-02 *	-2.727E-04	-4.072E-04	3.310E-04SLOW
.93	-1.50	-3.295E+01	-9.732E+01	-9.525E+01	-4.517E+01	-2.255E+02 *	5.692E-02 *	-1.433E-04	-1.535E-04	1.469E-04
.93	1.50	-3.295E+01	-9.732E+01	-9.525E+01	-4.517E+01	-2.255E+02 *	5.692E-02 *	-1.433E-04	-1.535E-04	1.469E-04
.93	2.25	-2.750E+01	-4.929E+01	-5.231E+01	-4.833E+01	-1.297E+02 *	5.699E-02 *	-7.399E-05	-5.709E-05	4.462E-05
.93	-3.00	-2.157E+01	-3.215E+00	-1.111E+01	-4.770E+01	-3.589E+01 *	5.698E-02 *	-3.978E-06	3.235E-05	-5.280E-05
.93	3.00	-2.157E+01	-3.215E+00	-1.111E+01	-4.770E+01	-3.589E+01 *	5.698E-02 *	-3.978E-06	3.235E-05	-5.280E-05
.93	3.75	-1.577E+01	4.246E+01	3.054E+01	-4.329E+01	5.724E+01 *	5.691E-02 *	6.621E-05	1.219E-04	-1.499E-04
.93	-4.50	-1.071E+01	8.916E+01	7.243E+01	-3.472E+01	1.509E+02 *	5.676E-02 *	1.363E-04	2.149E-04	-2.512E-04
.93	4.50	-1.071E+01	8.916E+01	7.243E+01	-3.472E+01	1.509E+02 *	5.676E-02 *	1.363E-04	2.149E-04	-2.512E-04
.93	5.25	-7.051E+00	1.363E+02	1.149E+02	-2.115E+01	2.462E+02 *	5.653E-02 *	2.081E-04	3.172E-04	-3.612E-04
.93	-6.00	-5.525E+00	1.916E+02	1.583E+02	-1.062E+00	3.444E+02 *	5.621E-02 *	2.793E-04	4.348E-04	-4.849E-04
.93	6.00	-5.525E+00	-1.731E+00	-2.063E+00	-1.062E+00	-9.313E+00 *	5.621E-02 *	2.793E-04	4.348E-04	-1.336E-03
3.93	0.00	-2.274E-01	-1.382E+02	-9.613E+01	4.570E-13	-2.345E+02 *	5.302E-02 *	-1.359E-04	-3.320E-04	3.116E-04SLOW
3.93	-1.50	-3.153E-01	-6.751E+01	-5.756E+01	-2.387E+01	-1.255E+02 *	5.333E-02 *	-1.013E-04	-1.432E-04	1.658E-04
3.93	1.50	-3.153E-01	-6.751E+01	-5.756E+01	-2.387E+01	-1.255E+02 *	5.333E-02 *	-1.013E-04	-1.432E-04	1.658E-04
3.93	2.25	-1.171E+00	-3.392E+01	-3.441E+01	-3.089E+01	-6.950E+01 *	5.349E-02 *	-6.790E-05	-6.563E-05	9.720E-05
3.93	-3.00	-2.170E+00	-5.357E-01	-9.420E+00	-3.376E+01	-1.213E+01 *	5.351E-02 *	-2.779E-05	1.367E-05	6.041E-06
3.93	3.00	-2.170E+00	-5.357E-01	-9.420E+00	-3.376E+01	-1.213E+01 *	5.351E-02 *	-2.779E-05	1.367E-05	6.041E-06
3.93	3.75	-3.057E+00	3.300E+01	1.635E+01	-3.233E+01	4.628E+01 *	5.349E-02 *	1.457E-05	9.231E-05	-7.602E-05
3.93	-4.50	-3.791E+00	6.717E+01	4.194E+01	-2.653E+01	1.053E+02 *	5.340E-02 *	5.523E-05	1.730E-04	-1.531E-04
3.93	4.50	-3.791E+00	6.717E+01	4.194E+01	-2.653E+01	1.053E+02 *	5.340E-02 *	5.523E-05	1.730E-04	-1.531E-04
3.93	5.25	-4.295E+00	1.024E+02	5.643E+01	-1.622E+01	1.646E+02 *	5.325E-02 *	9.039E-05	2.536E-04	-2.395E-04
3.93	-6.00	-4.433E+00	1.393E+02	6.869E+01	-1.112E+00	2.235E+02 *	5.304E-02 *	1.159E-04	3.520E-04	-3.189E-04
3.93	6.00	-4.433E+00	-1.566E+00	-2.072E+00	-1.112E+00	-8.121E+00 *	5.304E-02 *	1.159E-04	3.520E-04	-1.009E-03
1.97	0.00	2.752E-01	-1.047E+02					-5.304E-05	-2.722E-04	2.179E-04SLOW
1.97	-1.50	-4.577E-01	-3.710E+01					-4.215E-05	-1.233E-04	1.130E-04
[illegible]	[illegible]	[illegible]	[illegible]					[illegible]	-1.233E-04	1.130E-04

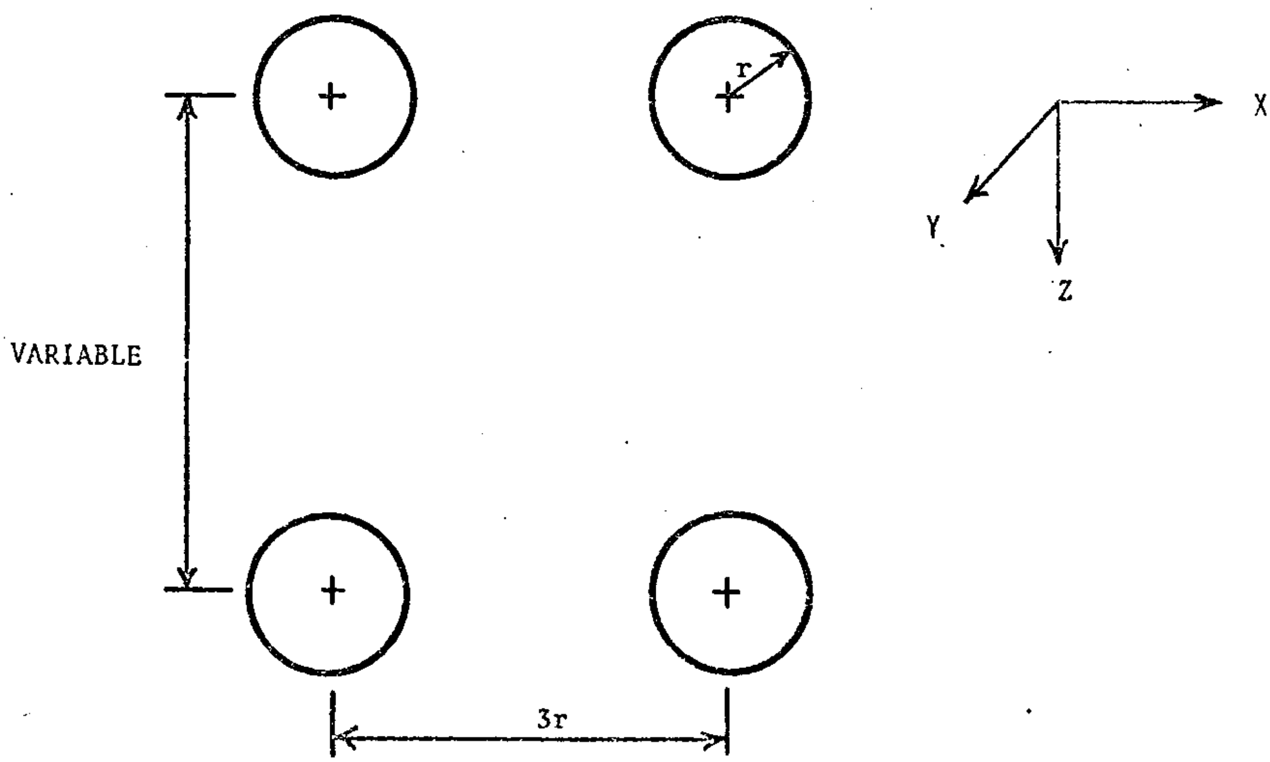
Figure 9(c) Output for Example Problem (PSAD2A)

```
                              POSITION NUMBER  1

                               LAYER NUMBER   1

                                COORDINATES

                          0.     X     0.     Y     0.     Z

           DISTANCE TO LOAD-AXIS( 1)                              THETA
               0.                                                 0.

DISPLACEMENTS
   RADIAL             TANGENTIAL            VERTICAL
 0.                     0.                   .6047E-01
  STRESSES
   RADIAL             TANGENTIAL            VERTICAL            RAD./TANG.          RAD./VERT.          TANG./VERT.
-.2840E+03              -.2840E+03          -.8011E+02
  STRAINS
   RADIAL             TANGENTIAL            VERTICAL            RAD./TANG.          RAD./VERT.          TANG./VERT.
-.4611E-03              -.4611E-03           .4902E-03

                         XX          YY          ZZ          YZ          XZ          XY          UX          UY          UZ
T O T A L  S T R E S S  -.284E+03   -.284E+03   -.801E+02                           0.
T O T A L  S T R A I N  -.461E-03   -.461E-03    .490E-03                           0.
T O T A L  D I S P L A C E M E N T                                                              0.          0.     [illegible]
```

Figure 9(d) Output for Example Problem (BISAR)



(a) Dual Wheel



(b) Dual Tandem

Figure 10 Typical Wheel Configuration

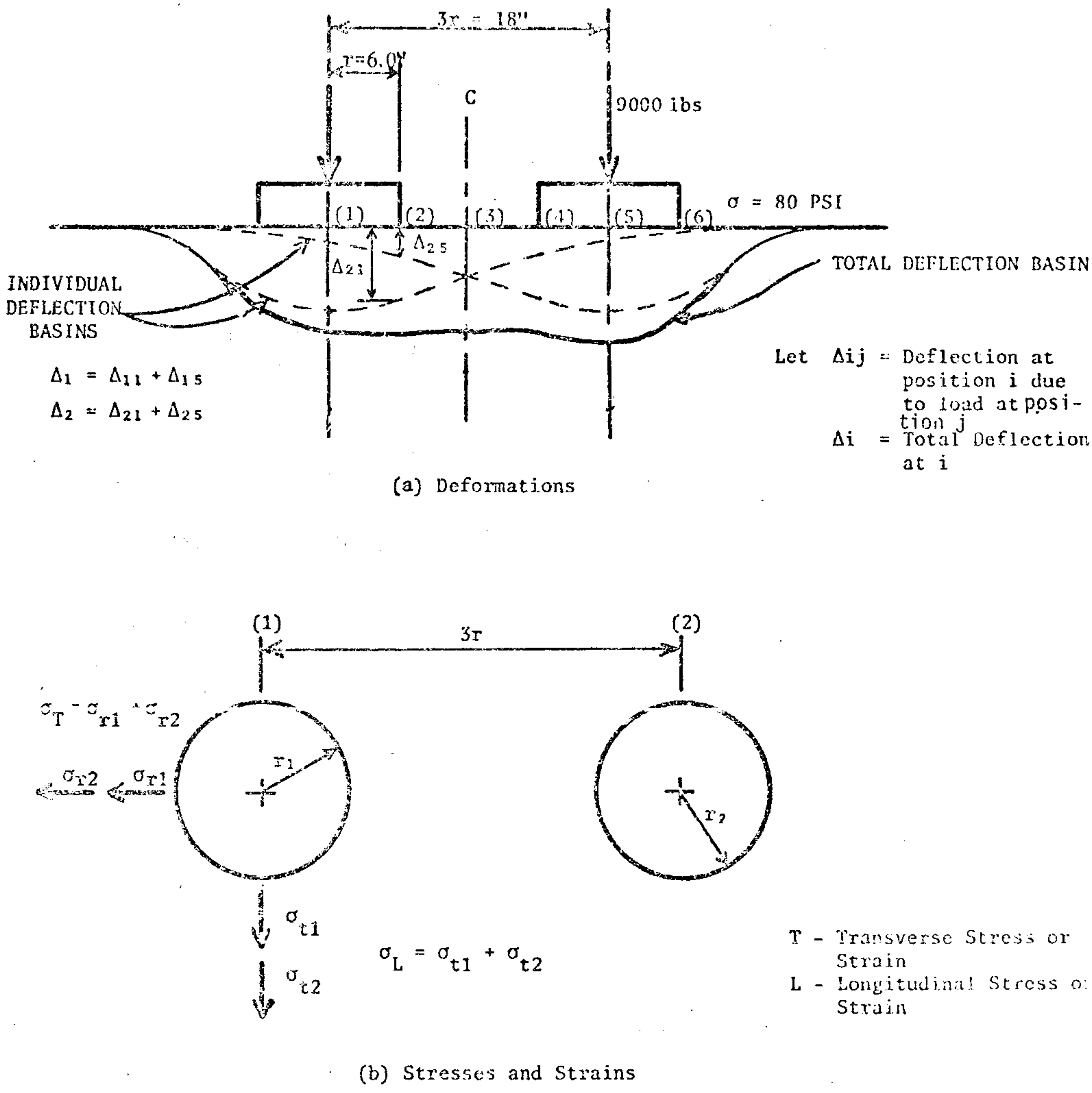


Figure 11 Example of Principle of Superposition for Dual Wheels

MATERIAL CHARACTERIZATION FOR ELASTIC LAYER ANALYSIS

During the past 10 to 15 years, people have shown considerable interest in the use of various elastic and viscoelastic layered system theories to predict the physical response of structural pavement sections. Use of such models has become more common because, by using high speed digital computers, one can rapidly solve more complicated layered system models than one could in the past. When an elastic layered theory is used to predict pavement response, one must either evaluate experimentally or estimate the modulus of elasticity and Poisson's ratio of each layer in the system. Therefore, the problem of determining the critical conditions for design and the pertinent material properties for each layer still remains.

The stress-strain properties of materials used in highway construction can vary greatly because of a number of factors including the stress state, pulse rate and duration, temperature, degree of saturation, density, age, and method of testing(1). Most routine design tests for pavements have been either static or been conducted at relatively low rates of strain. Use of theoretical design procedures, however, requires use of dynamic (generally) repeated load tests. A variety of different methods for the evaluation of the dynamic properties of pavement materials have been developed in recent years by researchers. Material characterization also has a great use in defining failure criteria for various distress modes such as repeated load (fatigue) cracking, permanent deformation, and thermal cracking effects.

Deacon(2) has carefully listed most of the material response variables as might be included in evaluation for pavement design. The major variables in Table 1 include: loading, environment, and mixtures. Kallas(3) presented the primary mixture variables as including: aggregates, binder, water, voids, and construction process. Although many of the variables included in Table 1 are probably reaching far beyond the intended scope os this discussion, they are at least indicative of the many factors that should be included in current, as well as future, design processes. Table 2 shows a wide range of test configurations which might be used to evaluate pavement material. Although the methods listed in this Table are primarily aimed at the properties of asphalt treated mixtures, they may be employed with subgrade materials as well.

CONVENTIONAL TESTS

These are the routine strength and deformation tests and are listed as follows:

- i) Plate Loading Test
- ii) Triaxial Test
- iii) California Bearing Ratio Test (CBR)
- iv) Stabilometer and Cohesiometer (R value)
- v) Bituminous Mixtures (S value)
- vi) Modulus of Rupture
- vii) Indirect Tensile
- viii) Marshall Stability

A detailed description of these tests is given by Yoder and Witczak(4), Chapter 8, Page 244-261.

The requirements for testing and evaluating of materials properties for the AASHO Interim Guide procedure, the Asphalt Institute Method, the State of California Method and Shell Method basically require only the use of currently standard procedures. The AASHO Interim Guide(5) is not based on laboratory results but, rather, has been developed from the AASHO road test and since has been modified through other experiences. The only material property concerned in this method is that of the soil support value S. In addition, material coefficients were developed for use in the thickness equation which indicated relative strengths of each material in the form of equivalencies. These values may be adjusted from those suggested based on testing local materials. Standard methods such as R-value, S-value, CBR, and others may be used for such purposes.

In the State of California (or Hveem) Method(6), the material tests are primarily related to the Hveem stabilometer and the cohesiometer. The R-value, as measured by the stabilometer, is the only requirement for the subgrade material. The design procedure also utilizes the tensile strength of various paving materials and these values have been determined using the cohesiometer and have since been modified into the gravel equivalent factor G_f. The third factor of note in this method is the expansion of the subgrade material.

The Asphalt Institute Method(7) does not require any tests for the asphalt treated material, although it is assumed that all of these materials meet certain minimum standards for design. The subgrade strength values may be determined by using either the CBR or the R value.

Similarly, for the Shell Design Method(8), all that is needed with respect to the materials, is an estimate or a measure of the CBR value from which an estimation of the elastic modulus is obtained.

NON-CONVENTIONAL TESTS

Although conventional tests are still currently useful, they may have limitations for newer materials, thick lifts, or special loadings. However, certain correlations of these and other values have been developed which may be useful in the interim period while more direct methods are being developed and standardized.

Some tests that have been used to define input parameters for layered theory solutions are listed as follows:

i) Resilient Modulus, M_R, for soils and some treated materials,

ii) Dynamic modulus from either triaxial or uniaxial tests

iii) Stiffness modulus from either flexural tests or nomographs based on standard tests

iv) Correlation with other tests.

In addition to obtaining material parameters for structural analysis, many of these newer test methods are suitable for design life predictions including fatigue and permanent deformation due to traffic.

Although the more recently developed Shell Method of structural design incorporates features which are intended to minimize two distress mechanisms (fatigue in the asphalt bound layers and rutting of the subgrade) the procedure is still based on assumed materials properties or at best, properties of the subgrade. As Monismith has suggested in his paper(9), when we tend to get more sophisticated in the area of preventing particular mechanisms of failure or distress, we must become more aware of the actual properties of the various materials within the section in order to develop a suitable model. As the engineer is required to change his design procedure to accommodate new materials, he must begin to look more carefully at the properties of individual layers within the structural system. In order to characterize a particular structural section which might include two or more materials, several thicknesses of a variety of loads and environments, a careful study of the individual materials properties must be made. In recent years, as newer theoretical analysis techniques have become available, which include elastic layered or viscoelastic layered systems, it has become increasingly evident that better tests or estimation procedures must be provided to characterize the material layers within the pavement. In other words, the theoretical aspects of pavement design have progressed somewhat further than our ability to provide suitable numbers or parameters for computational purposes. As can be noted from Tables 1 and 2, a variety of test procedures for a range of loads, stress-strain measurements, size and shape of specimens, etc., can become literally infinite. It has been recognized in recent years that there may be three basic modes of stress which the pavement engineer attempts to minimize; these are: (1) load associated fracture or fatigue (2) distortion or permanent deformation (3) fracture resulting from non-traffic-associated factors. The following discussion will include a brief description of the required apparatus and procedures for obtaining suitable design parameters appropriate for use with the first of these two subsystems.

Although materials properties that could be considered might include elastic, non-elastic, linearly elastic, viscoelastic, etc., this discussion will be limited to the elastic system; as this approach can now be utilized by the practicing engineer. The elastic layer system can be utilized most effectively to predict behavior rather than failure. Therefore, suitable parameters used in the elastic layer theory would tend to indicate more the behavior under moving wheel loads rather than an ultimate or "worst case" failure conditions. Accordingly, since the thick-lift technology concept is basically a two-layer system consisting of untreated subgrade and the asphalt treated layer; the following discussion will be broken down into these categories.

SUBGRADE OR UNTREATED SOILS [10]

Two basic properties of the soil or subgrade need be determined. These are: (1) the resilient modulus, M_R, which is analogous to Young's Modulus in the theory of elasticity and, (2) Poisson's ratio, which is also required for use in calculations. The modulus of soils can be determined basically in two ways: (1) directly by laboratory or field tests, (2) an estimate or correlation with other more conventional test methods.

Modulus Tests

The most suitable test for soil materials appears to be repeated load triaxial tests. Monismith has defined the resilient modulus and has indicated that for unbound materials it is stress dependent. Figure 1 illustrates a typical setup[11] for repeated load triaxial testing of soils. Although a triaxial test system may vary from unit to unit, the basic system should include at least the following features: (1) a large triaxial cell suitable for specimens up to 6" in diameter, (2) a loading system capable of providing a range of static and/or dynamic stress levels which is activated by a hydraulic system, an electrohydraulic system, or is mechanically controlled, (3) a timing device which permits some adjustment of the frequency and duration of load, and (4) suitable readout equipment for the type of deformation monitoring devices which are to be incorporated. Figure 2 shows at least one arrangement of the triaxial cell itself[12] indicating the position of the specimen within the cell, the arrangement of the load cell on top of the specimen, as well as the response measuring device which happens to be LVDT's clamped directly to the specimen. Two methods of measuring deformation of the specimen under repeated stress conditions appear to be most suitable at this time. The first of these utilizes a dial gage attached to the load piston on the outside of the triaxial cell, as shown in Figure 3. Although this approach provides a visual measurement of deformation, the indicated values actually include several sources of error, such as the effects of the interface between the specimen, end plates, porous stones, load cell, and the actual loading piston. Therefore, somewhat high values of deformation may result in lower resilient modulus values. Another approach utilizes dual LVDT's clamped directly to the specimen, providing a means of measuring the deformation over approximately the middle one-half portion of the specimen. When the specimen is subjected to dynamic loading, the deformation measured is more likely to be representative of that actually felt by the specimen. However, it must be recognized that considerable caution and extreme care must be exercised when incorporating these devices so as not to build in unknown errors that might be troublesome to evaluate.

Laboratory prepared samples should be selected in accordance with the expected field conditions. Specimens 4" in diameter by 8" high are often recommended as suitable for subgrade testing. Typically, compaction procedures may be selected from any one of normally used approaches including standard impact, gyratory, kneading or static compaction. The compaction method selected should be compatible with the field conditions and the moisture content or the moisture density relationships should be recognized in much the same way as compaction curves are developed for standard compaction construction control. The actual resilient modulus of subgrade materials may be far more dependent on factors such as soil suction rather than moisture content itself. Consequently, it may be appropriate to provide for testing of the soil suction value in conjunction with resilient modulus tests in order to provide a better relationship for the effect of moisture on a modulus.

The actual testing procedure may consist of applying a series of vertical stresses (σ_1) in a repeated mode and a range of confining pressure (σ_3) which is in the approximate range of those stresses expected in the field. In the triaxial test system a vertical stress is often designated in terms of the deviator stress (σ_d) or where $\sigma_d = \sigma_1 - \sigma_3$. Although the type of loading can be varied almost infinitely, Monismith(9), a common practice in recent test procedures has been to apply a near square wave repeated deviator stress for 0.1 sec. with approximately a two or three second rest period between load applications. For most applications σ_3 may be held constant while the vertical stress is repeatedly applied. However, more sophisticated test systems may provide the capability of also cycling σ_3 at the same time the vertical stress is cycled, thus more realistically representing in-situ field load conditions.

Depending upon the type of subgrade material being tested, the results may differ somewhat in their appearance when plotted as stress vs. modulus. For example, Figure 5 shows results of testing a clay subgrade material. Note that the resilient modulus is primarily dependent on the vertical applied stress and is virtually independent of the confining pressure. Figure 6, however, which is the result of testing a granular material, thows that the resilient modulus is almost wholly dependent on confining pressure.

Considering the foregoing discussion, one soon becomes aware that a single resilient modulus value for a particular soil is essentially non-existent. When analyzing a particular layered system for design, one must be somewhat aware of stress levels within the structure in order to judiciously determine a reasonable value for modulus. The design process, then, becomes a combination of stress analysis and materials characterization.

Correlation with Other Tests

From time to time, researchers have attempted to correlate the dynamic or elastic properties of soil materials with other standard or more conventional test procedures. One such approach as discussed by Monismith in relationship to the Shell method of design compares the subgrade modulus E_3 and the CBR value. This relation developed from field tests is:

$$E_3 = 1500 \times \text{CBR (psi)}$$

Since many of the early tabulated computations were developed assuming that Poisson's ratio was 0.5, the design charts were limited to these values. However, as more laboratory testing on subgrade materials has been accomplished, actual test values of Poisson's ratio have been added to our store of knowledge and it now becomes much easier to establish reasonable values. If actual testing utilizing lateral LVDT's in the triaxial test are not available, values ranging from approximately 0.30 to 0.50 can be used for the structural computations in the layered system.

Other more generalized correlations of several test procedures have been summarized by the Portland Cement Association and is shown in Figure 7. In situations where laboratory testing is not practicable, the engineer might indirectly select an appropriate modulus for the subgrade from other known test data.

ASPHALT TREATED MATERIALS (10)

For the purposes of this discussion, the materials considered are limited to those normally called asphalt concrete or asphalt treated base. Although other asphalt treated materials, such as emulsion treated mixtures, cutbacks, or combinations of these are part of the total asphalt structural section; they are not considered directly in this discussior. However, the techniques discussed are applicable to most pavement materials.

Laboratory procedures for the asphalt treated materials are somewhat similar for soils. However, slightly wider ranges of stresses test configurations, specimen types and measurement systems can be incorporated more successfully. In addition to the usual stress considerations, temperature also becomes an important factor. In recent years, as the newer design techniques required better and better evaluation of materials properties, many types of apparatus were developed at various research institutions. Most testing has centered around two basic approaches: (1) axial loading devices which were utilized to measure the dynamic stiffness or modulus as well as in some instances fatigue properties, and (2) dynamic flexural tests which generally utilized the beam principle with variations.

Specimen Preparation

Specimen preparation including the mixing and compaction of specimens for subsequent testing is not sufficiently different from the standard procedures to warrant an extensive discussion here. Basically, specimen shapes or types can be broken down into three categories: (1) cylindrical, (2) beams, and (3) plates or slabs. Cylindrical specimens may be compacted utilizing any one of the techniques discussed under the subgrade which include: kneading compaction, static, dynamic or combinations of these. Kneading compaction has been used very successfully for both cylinders and beams since the same machinery can be adapted to fabricate either of these types. In fact, it appears that the procedure for fabrication of asphalt concrete beams may soon become as ASTM standard. Other methods such as the Texas gyratory machine or compactor have been suggested as potentially useful devices since they are able to provide compaction as well as density control and some indication of their strength simultaneously.

Both beam and cylindrical specimens lend themselves well to field sampling. Coring machines and diamond saws have been used very successfully in obtaining large field-compacted specimens for subsequent testing in the laboratory. This approach may be very useful when it becomes desirable to compare either laboratory and field-prepared specimens or to obtain an indication of field behavior or performance.

Triaxial and Uniaxial Tests

Asphalt concrete or asphalt treated base specimens may be tested in the axial or triaxial mode and the test results can be used directly to obtain resilient or dynamic modulus, Poisson's ratio, and other design-oriented parameters which are in turn incorporated in layered system analysis. Direct axial loading, either compression or tension, is perhaps the simplest method of testing. Generally however, to get meaningful values for design, and in order to account for the time factors, the equipment should be capable of handling a variey of dynamic loads. The actual test can be used for fracture measurements as well as fatigue, creep, and modulus determination. In addition, temperature control must be provided.

Although many examples of direct axial testing could be considered, discussion here will be limited to several appearing to be representative. Perhaps one of the more direct approaches has been investigated by Kallas,[13] of the Asphalt Institute, and is shown in Figure 8. This procedure uses a cylindrical specimen and either a core or laboratory prepared specimen. Stress was limited to the compression or compression-tension stress reversals. The resulting response was measured directly by two axially-oriented strain gages as shown in the figure.

Another approach as discussed by Raithby and Sterling[14] from the Road Research Laboratory, utilizes a sawed parallelepiped specimen, also loaded in the axial direction as indicated in Figure 9. The axial stress can be applied either in tension or compression by means of the metal end plates which are cemented to either end of the specimen and attached directly to the loading machine. Response to the load is in this instance measured by four LVDT's attached to the end plates of the machine. Various sizes and shapes of specimens may be used in the direct axial test, another of which is illustrated in Figure 10,[Carré] and which also shows a slightly different arrangement for attaching the end plates. These end plates are glued directly to the asphalt concrete specimen and also connect by a mechanical device to a loading machine. Usually in tests such as these the applied load is monitored by a load cell attached directly to the end cap on the specimen so that the effect of slack in the loading system is minimized.

Although some researchers have found it expedient to use the axial test others have felt that providing a triaxial test configuration was more realistic in that many materials may be dependent both on axial and lateral stresses. In this approach one would note that a more realistic stress path under a moving wheel load can be achieved with a dynamic triaxial testing arrangement. A typical test might include conditioning a specimen under a given range of stresses to obtain a modulus and Poisson's ratio. This test could then be followed by a static test to failure to obtain the c and ϕ value as similar to that used in soil mechanics for shear strength criteria. Figure 11 shows a total testing system using a triaxial cell for characterizing pavement materials such as asphalt concrete and asphalt treated base. A basic system includes, in addition to the triaxial cell itself and its appurtinent loading devices, some arrangement for temperature control and accurate readout of the response to stress. In this apparatus the triaxial cell shown in Fig. 12 is capable of applying a wide range of vertical dynamic stress as well as a repeated load confining pressure similar to that experienced under a moving wheel load in the pavement structure. Thus very realistic stress paths may be developed in any of the 4" diameter by 8" high specimens. Temperature can be controlled very accurately to obtain a range of temperature dependent parameters within the material. Readout of test measurements can be provided in either analog form using a strip chart recorder or oscillograph. In some systems the actual response of the specimens may be digitized and may be analyzed directly while the test is under way.

Note item No. 2 in Fig. 12 are strain gages attached directly to the asphalt concrete specimen. The gages can be attached both axially and laterally to obtain modulus and Poisson's ratio values. Some researchers have found that with competent material such as asphalt concrete it may be more desirable to use strain gages rather than the LVDT's clamped to the specimen as shown in Fig. 4 as for soils. High modulus materials under very low stresses show very little strain response. Consequently, results obtained from clamped to the specimen may be difficult to evaluate under these conditions. Therefore, it is suggested that strain gages be utilized where possible and that the LVDT arrangement be limited to softer or less competent materials where strain gages are impractical. Figure 13 shows a close-up view of a strain gage which has been cemented directly to an asphalt concrete specimen. Note that care must be excercised to avoid using strain gages too small for the aggregate in the mix. Stress concentration and distortions may result if the gage is located near a large aggregate particle. Ideally, the gage should reach over a considerable length (at least 2" for an 8" specimen) of the specimen to average out these effects.

Figure 14 shows a sketch of the typical loading and control arrangement for both σ_1 and σ_3 stress application. Figure 15 indicates at least one arrangement for controlling temperatures during testing of asphalt treated materials. Generally, when asphalt treated materials are being evaluated, a modulus can be determined at several temperatures to establish the temperature susceptibility of these materials. One of the advantages of using lower (but representative) stress levels in repeated-load triaxial testing is that a single specimen can be used to measure the modulus over a range of stresses and temperatures without destroying the specimen or distorting the data.

Variations of the triaxial test system have been developed by researchers for a variety of pavement materials. Figure 16 shows the apparatus developed and reported by Shackel [15] in Australia. In this test system, considerable effort was expended in developing a complex stress application procedure which more directly simulates the in-place situation. Both soils and other treated materials can be tested successfully with this hydro-mechanical device.

As might be expected, with any of these triaxial testing devices, no single value of modulus or Poisson's ratio is obtainable. The results will depend upon the many factors of stress, rate of loading, temperature, and others. Figure 17 shows the results of a typical test for asphalt concrete. As can be noted in the figure the dependency of resilient modulus on lateral pressure is almost negligible. However, the deviator stress or the applied vertical stress makes considerable difference in modulus. In the figure a range of about 120,000 psi in modulus exists when the deviator stress ranges between 5 and 40 psi. In addition to stress, temperature has a considerable effect as indicated in Figure 18[43] where the resilient modulus and Poisson's ratio measured over a range of temperatures which would be expected in a pavement and for a given state of stress and time of loading.

Flexural Tests

It would appear that the flexural tests might be more suitable for asphalt bound layers in the pavement system since it may more realistically tend to represent the state of stress experienced in the field. Several approaches have been utilized to similate field conditions in past research projects. In addition, several researchers have attempted to relate or correlate the results of modulus measurements using both flexural and triaxial dynamic test methods.[17][18] In general, it would appear that these modulus values would differ by approximately a factor of two or three. In the axial or triaxial load, stresses are normally either compression or tension at any one time, while in the flexural test, extreme fiber stress varies from compression at the top to the tension at the bottom. In general, it has been shown that the modulus measured either in compression or tension does not vary substantially.

Flexural specimens generally vary in both size and shape as well as in cross-sectional area and length, depending upon the testing apparatus being used. In many instances, the specimen may be rectilinear in shape, but in others, circular or prismatic specimens have been used which have been used which have been "necked down" to provide a known plane of failure. Others have included cantilever beams of varied cross-section, resulting in a trapezoidal shape which provides a constant bending moment throughout the length of the specimen. All of these procedures seem to give similar results; again probably within a factor of two or three. One limitation of the flexural test procedure, however, is that very soft materials cannot be tested because of the lack of integrity so that they may fail under their own deadweight. In addition, Poisson's ratio is more difficult to measure.

One of the earlier methods reported to this association in 1961 by Monismith, et al, utilized beams in repeated flexure resting on a continuous bed of springs. A diagram of this apparatus (19) is shown in Figure 19, which uses beams 2" deep by 3" side and 12" long. Later work at the University of California and a modification of the apparatus initially developed by Deacon(20) and later modified by others at the University of California, is illustrated(21) in Figure 20. This machine utilizes specimens 1½" square in cross-section and approximately 15" long. The beam is simply supported at the ends and is loaded at the third points so that a constant bending moment is developed between the load points. Repeated loads are applied by a Bellofram type loading piston as shown in the bottom of the photo. Deformation is monitored by an LVDT which is attached to the side of the specimen so that total deflection is measured with each repetition of load. In order to simulate an elastic subgrade, the specimen is returned to its original horizontal position at the conclusion of each loading pulse. Similar devices have been used by others such as the Asphalt Institute and Chevron Research Corporation.

Another approach incorporating a rectilinear beam has been developed by Majidzadeh, et al, at Ohio State University. Although a different loading device was used, a specimen arrangement is shown in Figure 21. In this case, it would appear that the beam could be analyzed as a beam on an elastic foundation. A 3_{xy} by 24" long asphalt concrete specimen overlies a soft rubber base which in turn overlies a rigid base. A repeated load is applied at the center point of the beam.

A variation of the beam test in repeated flexure has been used by Saunier[22] of the Shell-Berre Laboratories in France. This test is a cantilever beam held rigidly at one end and loaded repeatedly at the other end. The beam itself, rather than being rectilinear, is trapesoidal in shape, so that the resulting bending moment is constant along the length of the beam. Failure can theoretically occur at any point in the beam. Figure 22 illustrates one of the sawed beam samples with the two special end caps. The whole beam assembly is immersed in a fluid so that a constant temperature is provided through the duration of the test.

Still another device has been developed at the University of Nottingham by Pell and Taylor.[23] In this case, as shown in Fig. 23, the beam has been "necked down" at the center to a smaller diameter than the ends, which are connected to the loading mechanism of the testing machine. The circular specimen then spins about its linear axis while the load is applied at the top of the specimen so that at all times the beam is stressed in a cantilever fashion. Any one point on the specimen is repeatedly reversed from compression to tension throughout the duration of the test. Also, the specimen is submerged in water for accurate stiffness and fatigue determination over a range of appropriately controlled temperatures.

The flexural test has been used primarily to measure the stiffness or modulus of asphalt mixtures as well as fatigue life under large numbers of repeated loads. Figure 24 shows an experimentally determined relationship between the average stiffness modulus (calculated using the center deflection of the specimen at the first load application) and the extreme fiber bending stress.(24) Also to be noted from this figure is that the stiffness modulus is a function of stress level in a manner similar to that shown as for the triaxial state condition. All of the flexural beam tests give very similar results with respect to stiffness for a variety of asphalt-bound materials. Interpretation of test results may vary from system to system however, with judicious attention paid to test configuration, beam size and relationship of beam size to aggregate size. Very similar results, at least within a factor of two or three, can generally be measured. Kallas and Kingham(25) at the Asphalt Institute have recently conducted a large series of repeated load beam tests in fatigue and found considerably less scatter when larger beams (3" sq. in cross-section) are used rather than those of smaller dimensions. The scatter reduces the error considerably when interpreting the test results in terms of design life. In the advent of utilizing larger aggregates in asphalt treated base courses it may become necessary or at least desirable, to use larger beams for these tests, thereby requiring an adaptation of existing testing machines.

OTHER TEST METHODS

The test procedures briefly described above, generally incorporate stresses, strains, and deformations in the same order of magnitude as those experienced in the field. In the interest of simplicity and expediency it may be desirable to evaluate asphalt bound materials using indirect procedures which may give moduli or other parameters as measured from indirect test methods. For example, sonic tests such as those used by Goetz[26] and others[27,28] require superpositioning shifts of data to reduce the modulus to periods similar to those existing in pavement. A recent paper by Stephenson and Manke[29] shows results of ultrasonic moduli testing of asphalt concrete with the general conclusion that the range of modulus values measured was appropriate with respect to a range of temperatures and other mix properties. However, they were somewhat higher than those expected under realistic field loading. Figure 25 shows the general test arrangement while Fig. 26 includes typical results. Of considerable importance in this sort of testing is the difficulty in making certain that the measured fundamental frequency of the system is not really a harmonic. In order to make certain of this, moduli must be determined over a wide range of temperatures, which may be impractical in the normal laboratory situation. There is some discrepancy in the literature regarding the sensitivity of the sonic modulus test to determine the difference between asphalt mixes with varying viscosity or amount of asphalt. The work of Goetz[26] or Blaine and Burlot[28] tend to indicate a general insensitivity while Shook and Kallas[30] using lower frequency dynamic tests found this to not be the case. The data shown in Fig. 26 as developed by Stephenson and Manke, however, shows a definite decrease in modulus with temperature, and apparent difference with regard to the asphalt content. To date, it has not been demonstrated that these values can be effectively utilized in evaluating the materials or for incorporation in a multi-layered elastic system for structural design.

Schmidt[31] has recently reported on the development of an indirect tensile testing device for measuring determination of the modulus of asphalt bound materials. He calls this device the diametral resilient modulus device, a photo which is shown in Fig. 27. This device uses a standard 4" diameter by 2½" high specimen fabricated using either the Marshall apparatus, the Hveem kneading compactor or cores removed directly from the pavement. A repeated load is applied across the diameter, placing the specimen in a state of tensile stress along the vertical diameter. LVDT's can then measure the lateral displacement of the specimen under this load. Schmidt has shown that the modulus can be determined using the following equation:

$$E = \frac{P}{t} \left(\frac{\mu + 0.2734}{\Delta} \right)$$

where E = elastic modulus

P = applied vertical load

t = thickness of specimen

ν = Poisson's ratio, and

Δ = lateral deformation as measured by the LVDT's

Although this device has seen limited use to date, it shows considerable promise in providing a very simple yet suitable method of determining a resilient modulus for design purposes. In addition, a device such as this or similar to that developed at the University of Texas, Austin, could possibly be used for fatigue evaluation.

CORRELATION WITH CONVENTIONAL TESTS

The tests discussed above, are necessarily somewhat specialized, and therefore require considerable effort and equipment in order to effectively evaluate the modulus or strength properties of paving materials. It would appear that a procedure whereby the engineer could correlate some of these newly recognized engineering properties with more conventional or standard tests would be desirable. Several of these procedures have been adapted for use in structural design in order to provide a means of estimating resilient modulus for computational purposes. The following brief discussion of some of these approaches at least indicates that there is some potential for correlation. However, there are limitations and these correlations should be used for estimating only when actual test data are not available.

Nijboer[32] has suggested a method for predicting or estimating the asphalt mixture stiffness (analogous to resilient modulus) as follows:

$$S_{60^\circ C}\text{, 4 sec. } (kg/cm^2) = 1.6\ \frac{\text{Marshall stability (kg)}}{\text{Marshall flow (mm)}}$$

Nijboer noted that it was necessary to adjust the test temperature in such a way that a loading time of 4 seconds will cause a response similar to the actual combination of time and temperature on the road. In other words, he recognized the validity of the time-temperature superposition principle.

McLeod[33] also developed an estimating procedure for the asphalt mixture stiffness as follows:

$$S(\text{psi}) = 40\ \frac{\text{Marshall stability (lbs)}}{\text{Marshall flow (units of 0.01 in.)}}$$

He suggested that this stiffness relationship was valid for a range of mixtures.

Finn, et al, [34] working with data obtained from extracted cores of emulsion treated base layers have developed an expression relating the sand fraction of the aggregate and the penetration of the asphalt to the dynamic modulus. This relationship is as follows:

$$\text{Log}_e\ M_r\ (\text{modulus} \times 10^{-3}\text{in. psi}) = 1.86 - 0.016\ (\text{penetration, .01 in.}) + 0.047\ (\text{density, pcf}) + 2.58\ (\text{sand fraction of agg.})$$

$R = 0.659$ S.E. $= 0.680$

where: sand fraction = % passing No. 4, retained on No. 200, expressed a decimal.

R = coefficient of correlation

S.E. = standard error of estimate

Shook and Kallas[30] investigated the relationship between the flexural modulus and several routine or standard tests including the Marshall stability, Marshall flow value, Hveem stability, Hveem cohesiometer value, ultimate tensile strength, elongation in direct tension, and elongation in indirect tension or the split-tension test. Even though several corrections were made in regard to air-voids in the specimens, no practical degree of correlation was found between the resilient modulus and most of these other conventional test values. However, fairly good correlations were obtained using the results of the ultimate tensile strength or the Marshall stability/flow ratio as related to the resilient modulus. These relationships are as follows:

a) $\text{Log}_{10} |E^*| = 0.983861 + 0.00351866(U) - 0.052137(V)$

$R = 0.744$, S.E. $= 0.284357$

b) $\text{Log}_{10} |E^*| = -0.124262 + 1.25469(K) - 0.0616215(V)$

$R = 0.900$, S.E. $= 0.151416$

where:

U = ult. tensile strength (2"/min)(psi)

V = % air voids for modulus specimen - % air voids for test specimen

$$K = \frac{\text{Log}_{10} \text{ Marshall stability (lbs)}}{100 \times \text{flow (.01")}}$$

E^* = dynamic modulus (@ 4 cps)(psi)

R = coefficient of multiple correlation

S.E. = standard error of estimate

Hadley, et al,(35) investigated possible correlations between the stiffness or resilient modulus and Poisson's ratio determined for the indirect tension test at 75°F and a standard beam test at 140°F. They found reasonable acceptable correlations for a general range of test conditions for (a) the modulus of elasticity and cohesiometer values, and (b) Poisson's ratio and stability as follows:

a) $E = 0.613 \times 10^5 + 3.305 \times 10^2 (C)$

95% confidence limits = $\pm 1.434 \times 10^5$

b) $\nu = 0.470 - 0.0047(S)$

95% confidence limits = ± 0.184

where:

E = modulus of elasticity (psi)

= Poisson's ratio

S = Hveem stability

C = Hveem cohesiometer value

95% confidence limits = ± 2{standard error of estimate} by van der Poel. These may be based on different viscosity relationships, sources of the asphalt, or on other factors. Also, the asphalt content as used in some of the leaner asphalt treated bases may deviate somewhat from those predicted by Hukelom and Klomp. Figure 29 shows theoretical stiffness curves based on the nomograph, and superimposed on these are the modulus values actually determined experimentally. Note that the three asphalt contents indicated are relatively low as compared to asphalt concrete mixtures, and that temperature has less effect on the stiffness than would be the case for higher asphalt contents, in which case the asphalt played a larger role in the overall stiffness of the mixture.

DESIGN LIFE CONSIDERATIONS

Monismith, in this Symposium discussion has considered the design thickness and the design life factors utilizing the conventional as well as more theoretical design techniques. These include the use of materials properties developed from laboratory procedures such as those discussed above. As might be expected, considerable more work will be required to evaluate and standardize the suitable ranges of stresses, temperatures, times of loading, and other factors which should be incorporated in the total design procedure. Although materials characterization and experimental procedures appear very complex, considerable progress has been made in recent years and researchers are beginning to recognize the more important factors in the total design system. Eventually, it would appear that the procedures will become simplified as the sensitivity to various material parameters becomes better known and some of them can be reduced or eliminated.

One or more of the tests described above may eventually be required in the test system to evaluate properties of materials required in the three structural design subsystems discussed earlier. Each of these subsystems may require a separate test or a combination of tests in order to characterize the important properties. For example, the two fracture sybsystems appear to be most readily tested in the tension mode, since the tensile strength plays a dominant role in the failure or distress mechanisms. Permanent deformation or distortion may be best simulated by the triaxial type of test where confining pressures similar to those experience in the field can be accommodated and can result in realistic deformations. Associated with these tests will be the development of meaningful criteria and limits for both the testing procedure and the structural design process.

Although the several tests and procedures discussed in this presentation were not intended to be complete, they at least represent some of the efforts being made to better accommodate design procedures for all-asphalt or thick-lift construction.

TABLE 1

VARIABLES AFFECTING MATERIAL RESPONSE

I. LOADING VARIABLES

A. Stress history (nature of prior loading)

1. Non-repetitive loading (such as preconsolidation)
2. Repetitive loading

a. Nature

(1) Simple
(2) Compound

b. Number of repetitive applications

B. Initial stress state (magnitude and direction of normal and shear stresses)

C. Incremental loading

1. Mode of loading

a. Controlled stress (or load)
b. Controlled strain (or deformation)
c. Intermediate modes

2. Intensity (magnitude and direction of incremental normal and shear stresses)
3. Stress path (relation among stresses - both normal and shear - as test progresses)
4. Time path

a. Static

(1) Constant rate of stress (or load)
(2) Constant rate of strain (or deformation)
(3) Creep
(4) Relaxation

b. Dynamic

(1) Impact
(2) Resonance
(3) Other

(a) Sinusoidal (rate of loading is variable)
(b) Pulsating (duration, frequency, and shape of load curve are variables)

(con't on next page)

TABLE 1 (con't)

VARIABLES AFFECTING MATERIAL RESPONSE

5. Type of behavior observed
    a. Strength (limiting stresses and strains)
    b. Deformability
6. Homogeneity of stresses
7. Drainage (drained or undrained)

II. MIXTURE VARIABLES

A. Mineral particles
1. Maximum and minimum size
2. Gradation
3. Shape
4. Surface texture
5. Angularity
6. Mineralogy
7. Adsorbed ions
8. Quantity

B. Binder
1. Type
2. Hardness
3. Quantity

C. Water
1. Quantity

D. Voids
1. Quantity
2. Size
3. Shape

E. Construction Process
1. Density
2. Structure
3. Degree of anisotrophy
4. Temperature

F. Homogeneity

III. ENVIRONMENTAL VARIABLES

A. Temperature

B. Moisture

C. Alteration of Material Properties
1. Thixotropy
2. Aging
3. Curing
4. Densification

(Ref. 1)

TABLE 2

TEST CONFIGURATIONS

I. Tension A. Uniaxial tension B. Indirect tension 1. Splitting tension 2. Cohesiometer	II. Compression A. Unconfined, uniaxial compression B. Triaxial compression 1. Open system a) Isotropic Compression b) Conventional triaxial compression (1) Normal (2) Vacuum (3) High pressure c) Box with cubical specimen 2. Closed system a) Oedometer b) Cell c) Hveem stabilometer	III. Flexure A. Rotation 1. Rotating 2. Non-rotating B. Loading 1. Cantilever 2. Simple beam a) Point supports b) Uniform supports
IV. Direct shear A. Direct shear (rigid split box) B. Double direct shear C. Uniform direct shear (rigid caps with confined rubber membrane and split rings for lateral restraint) D. Uniform strain direct-shear (hinged box) E. Punching shear	V. Torsion A. Pure torsion B. Triaxial torsion C. Specimen shape 1. Solid cylinder 2. Thick-walled, hollow cylinder	VI. Indirect A. Penetration B. Squeeze tests C. Marshall stability D. Angle of repose E. Others

(Ref. ..

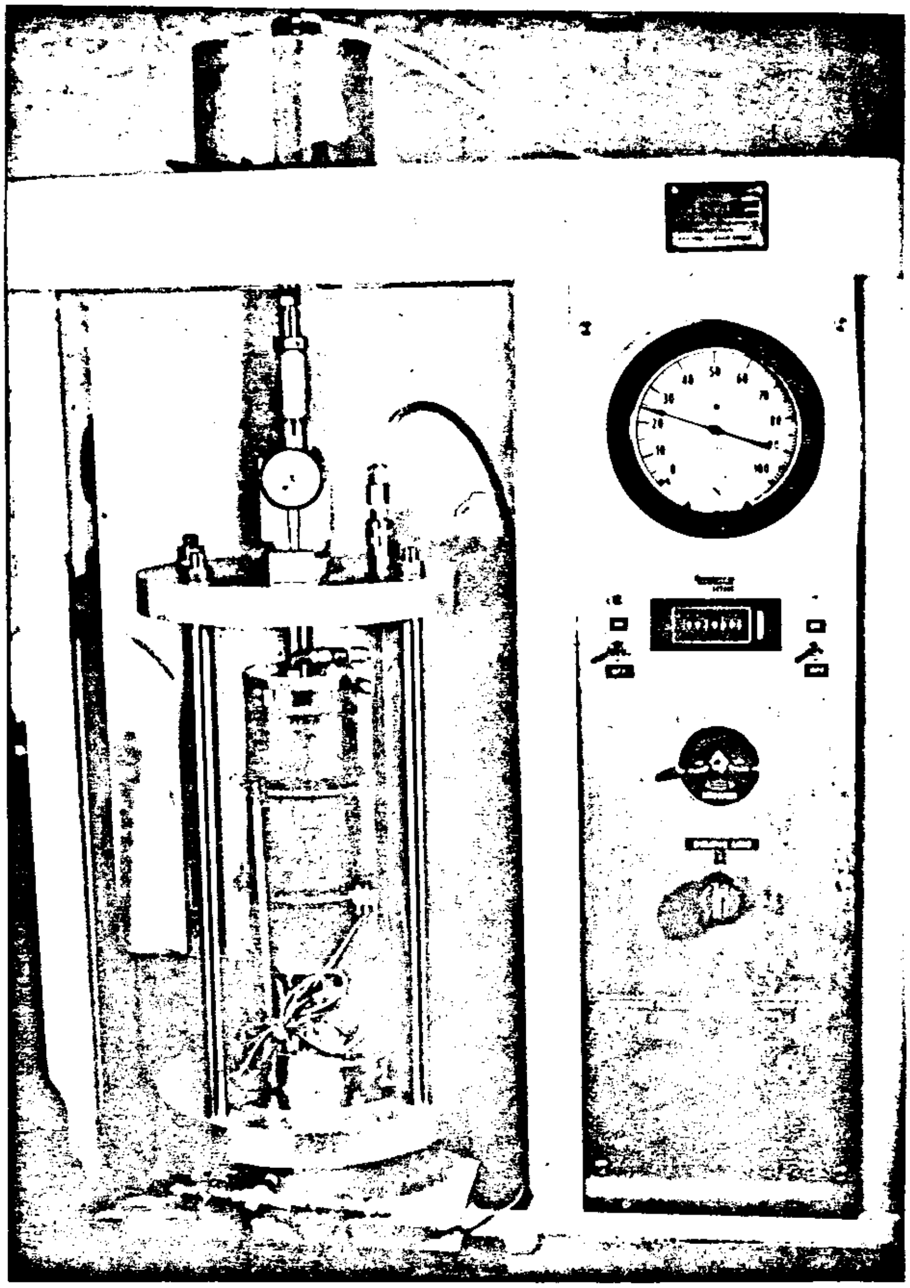


FIGURE 1 - CLOSE UP OF TRIAXIAL CELL AND CONTROL PANEL FOR REPEATED-LOAD TEST SYSTEM

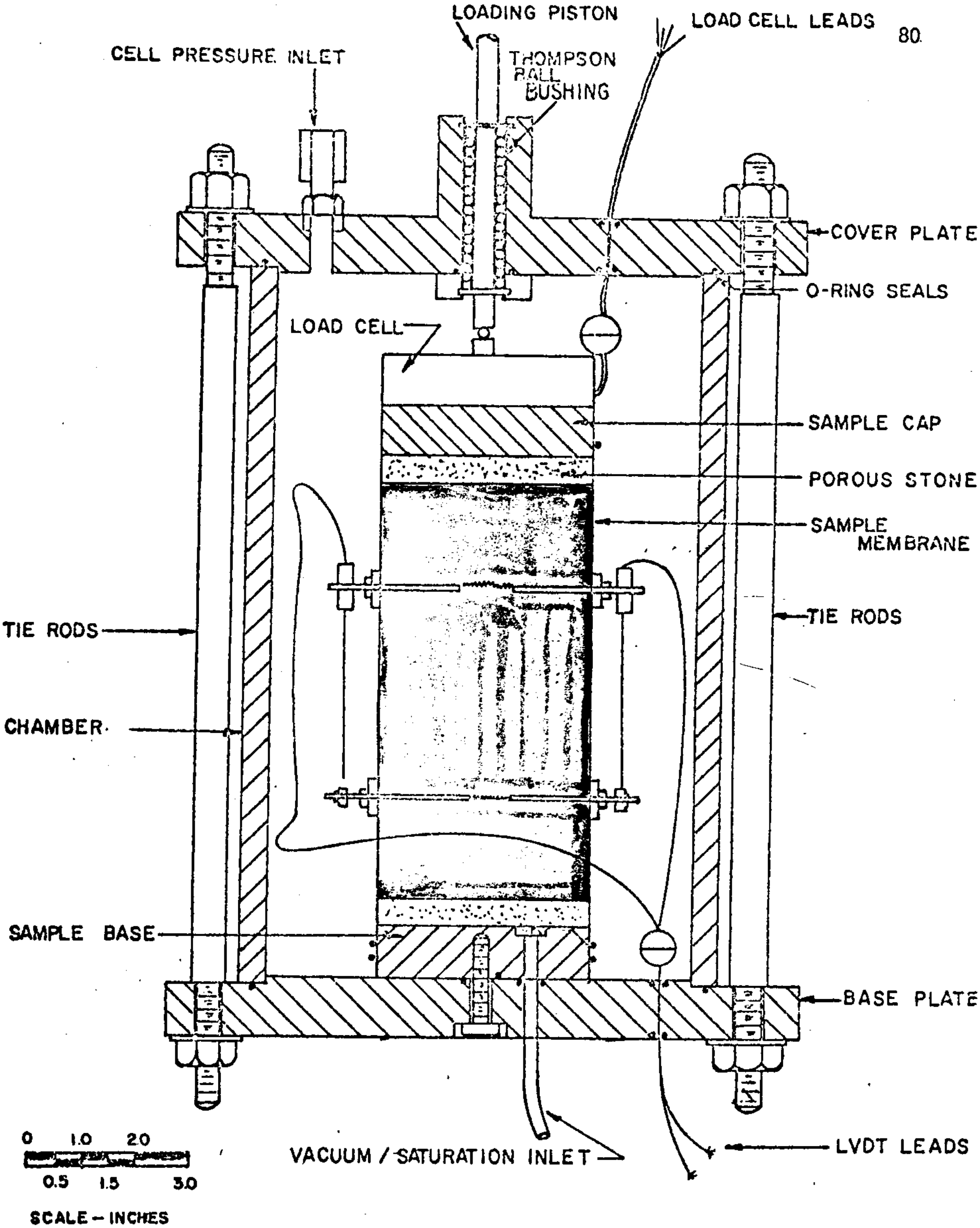


FIGURE 2 - TRIAXIAL CELL FOR RESILIENT MODULUS
TESTING OF SUBGRADE SOILS

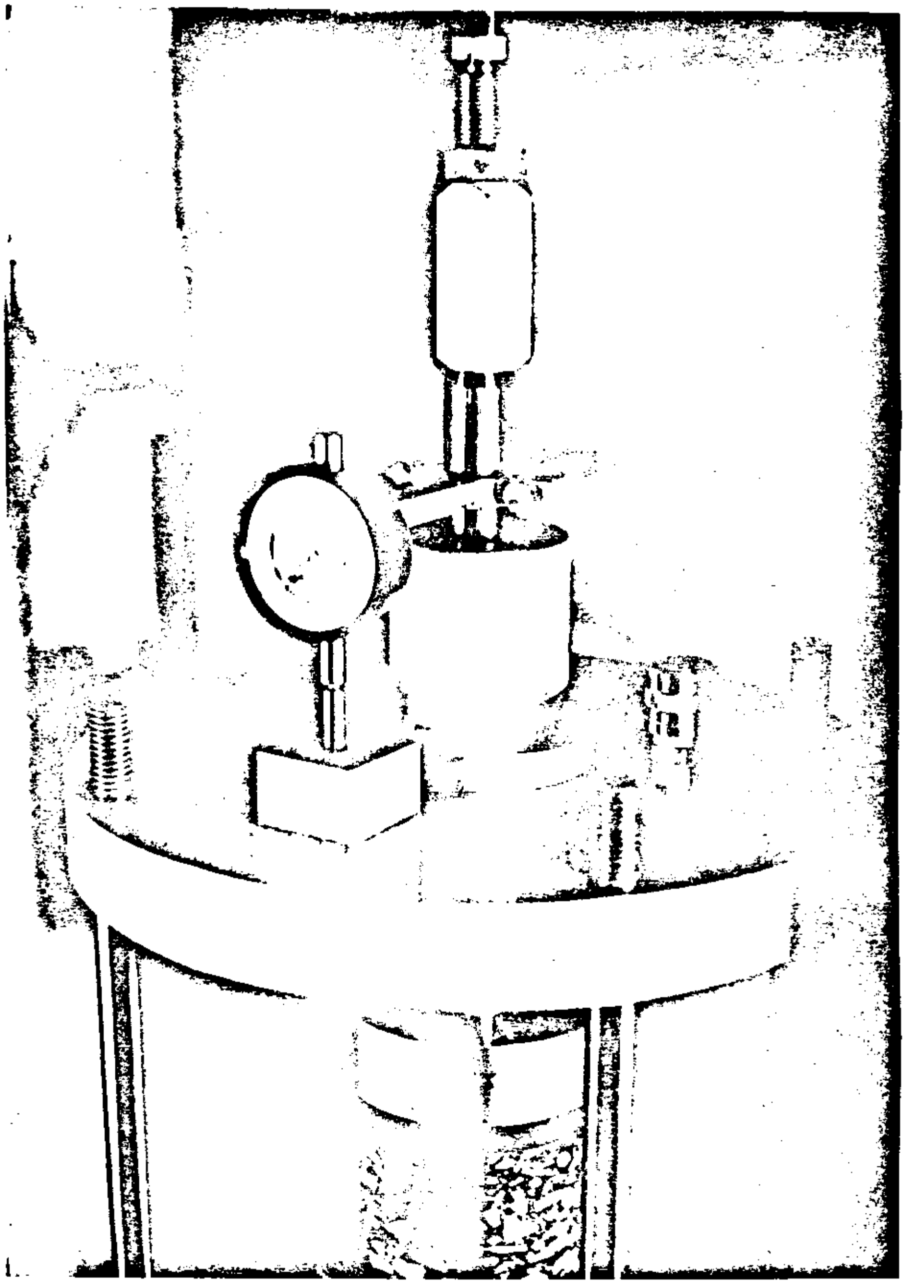


FIGURE 3 - DIAL GAGE ARRANGEMENT FOR MEASURING
DEFORMATION ON OUTSIDE OF TRIAXIAL CELL

FIGURE 4 - CIRCUMFERENTIAL CLAMP AND LVDT's FOR MEASURING AXIAL DEFORMATION UNDER REPEATED LOADING. (35)

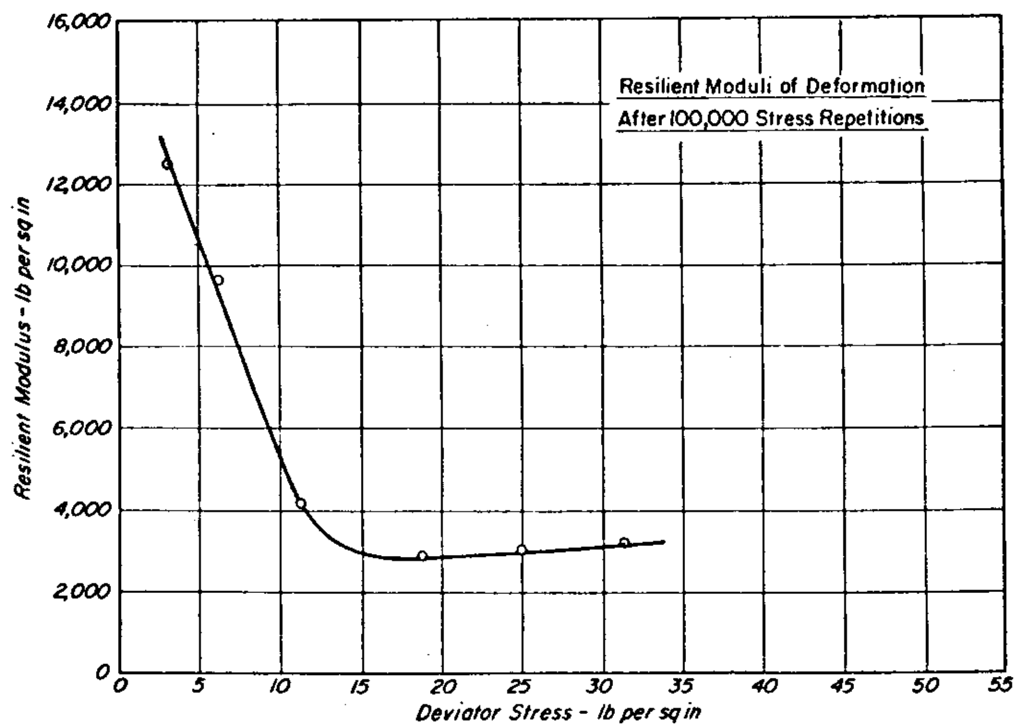


FIGURE 5 - EFFECT OF DEVIATOR STRESS ON RESILIENT MODULUS OF FINE-GRAINED SOIL. (39)

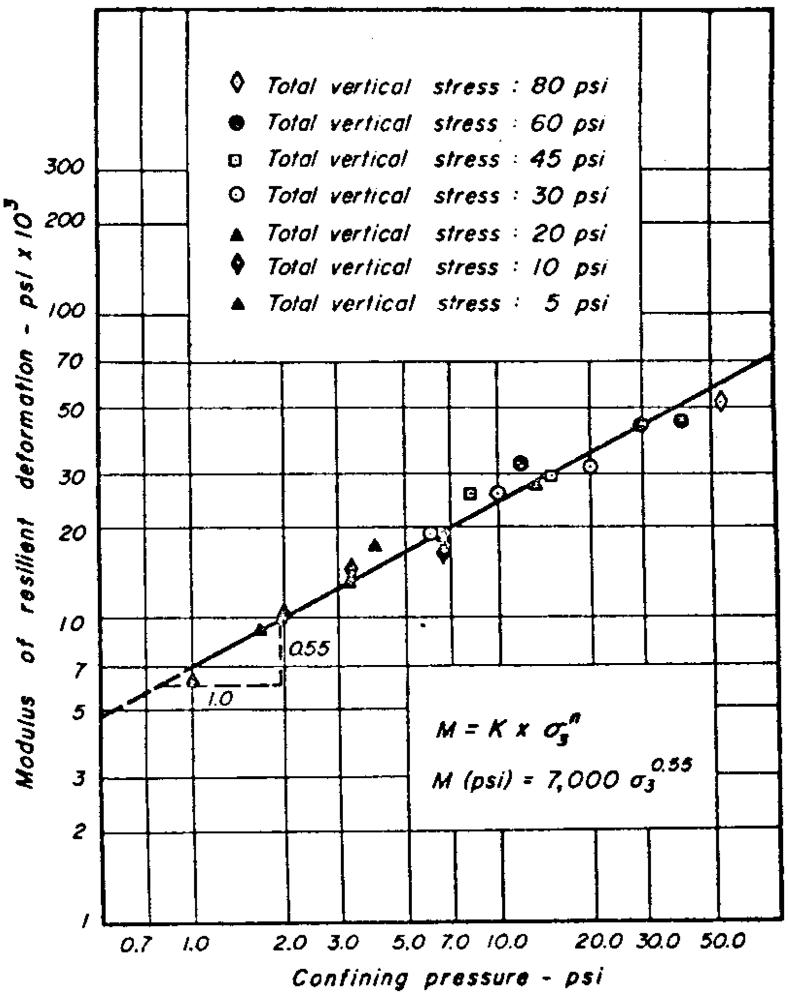


FIGURE 6 - EFFECT OF CONFINING PRESSURE ON RESILIENT MODULUS FOR GRANULAR MATERIALS. (39)

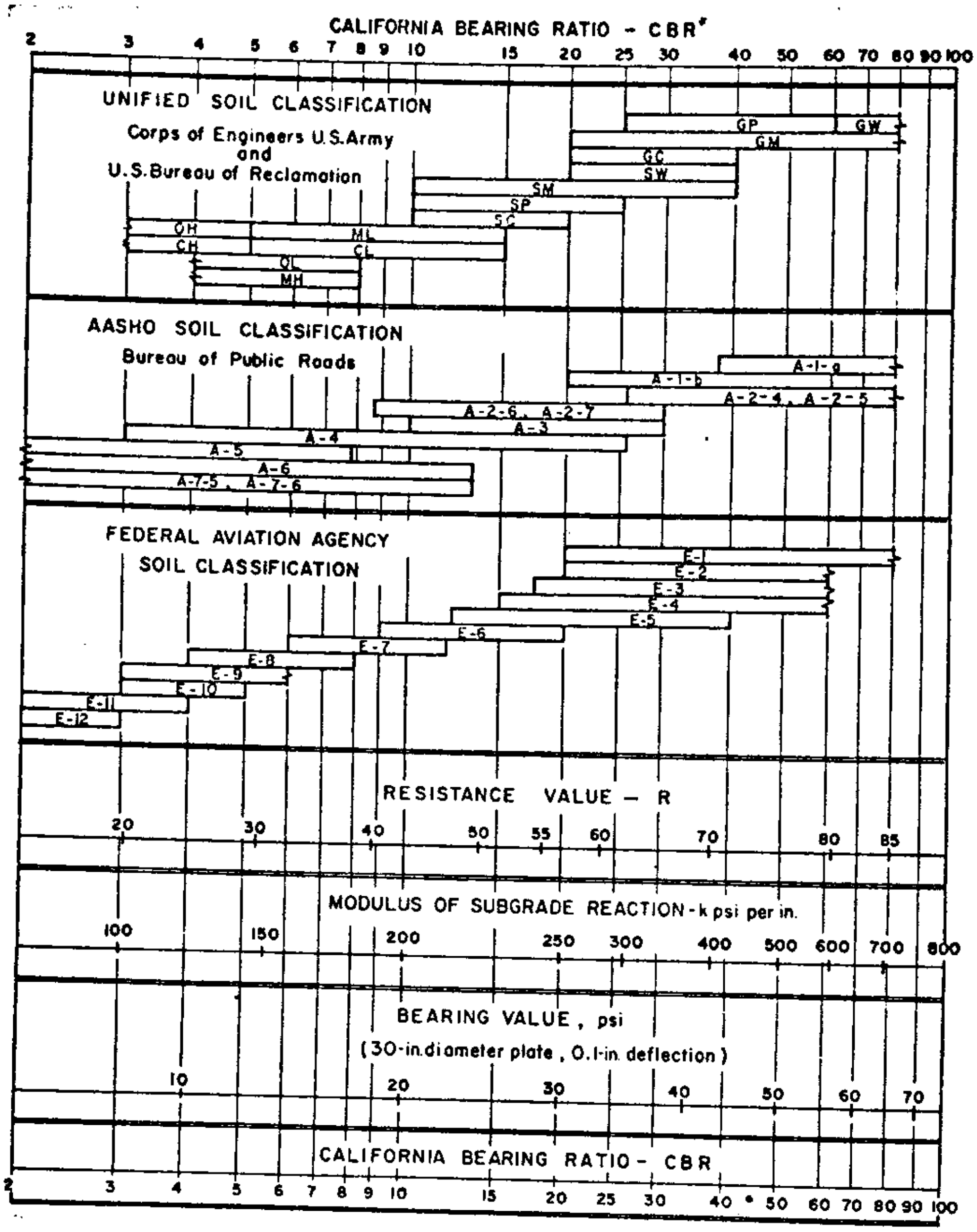


FIGURE 7 - APPROXIMATE RELATIONSHIP AMONG SEVERAL TYPES OF SOIL CLASSIFICATION AND TEST VALUES. (40)

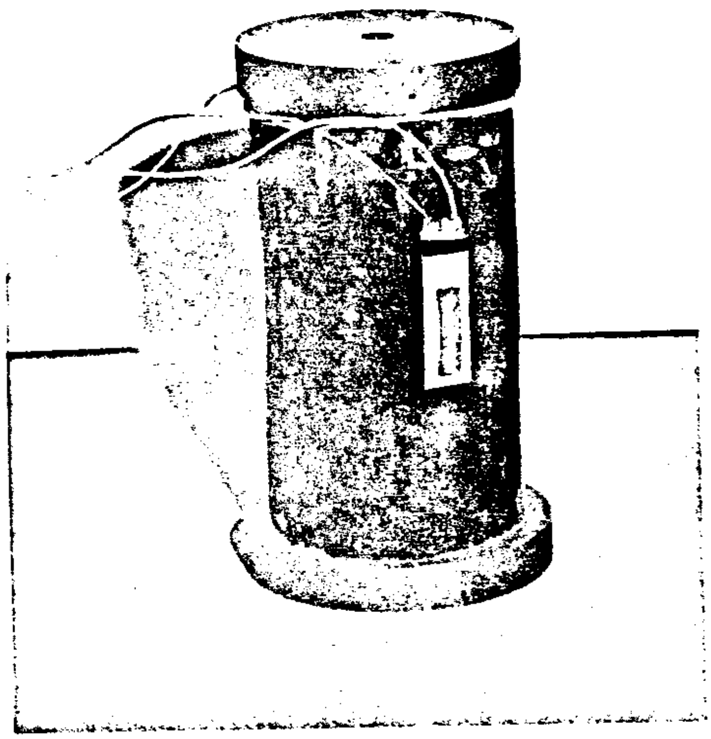


FIGURE 8 - ASPHALT CONCRETE SPECIMEN WITH STRAIN GAGES ATTACHED FOR DYNAMIC AXIAL COMPRESSION-TENSION TESTING (8)

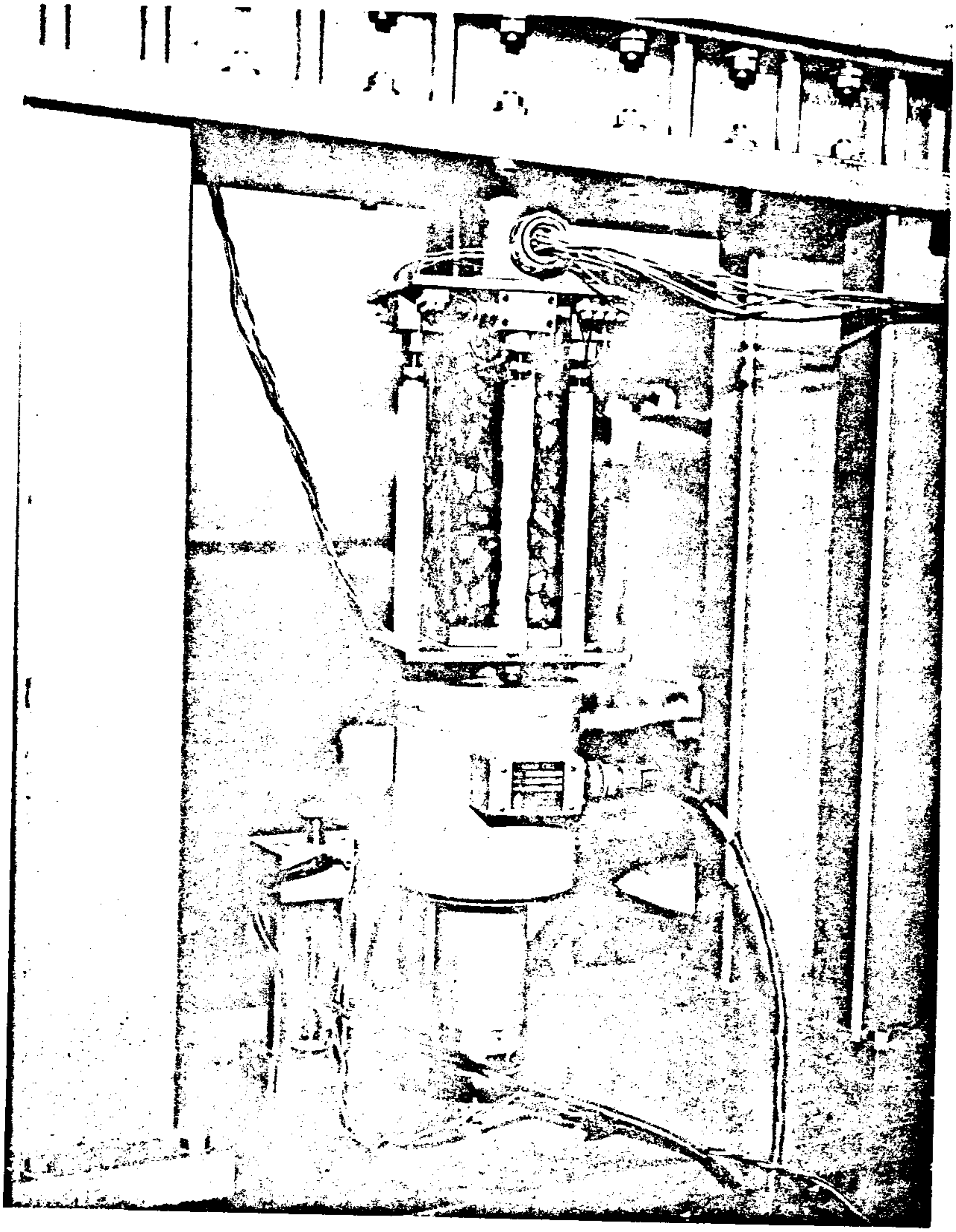


FIGURE 9 - SAWED ASPHALT CONCRETE PRISM SET-UP FOR DYNAMIC AXIAL COMPRESSION-TENSION TESTING. (9)

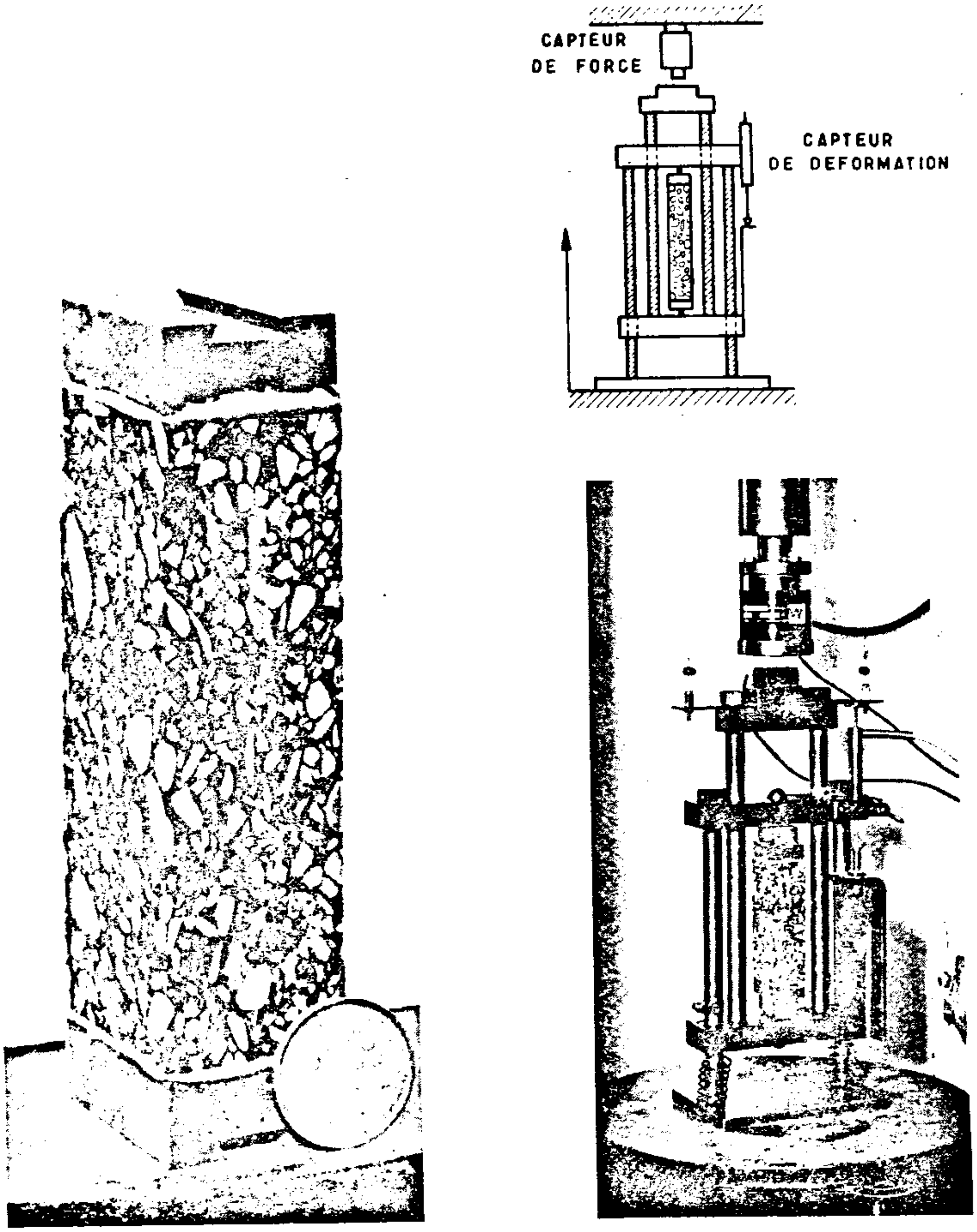


FIGURE 10 - ANOTHER ARRANGEMENT FOR AXIAL TESTING
OF SAWED ASPHALT CONCRETE SPECIMENS (37)

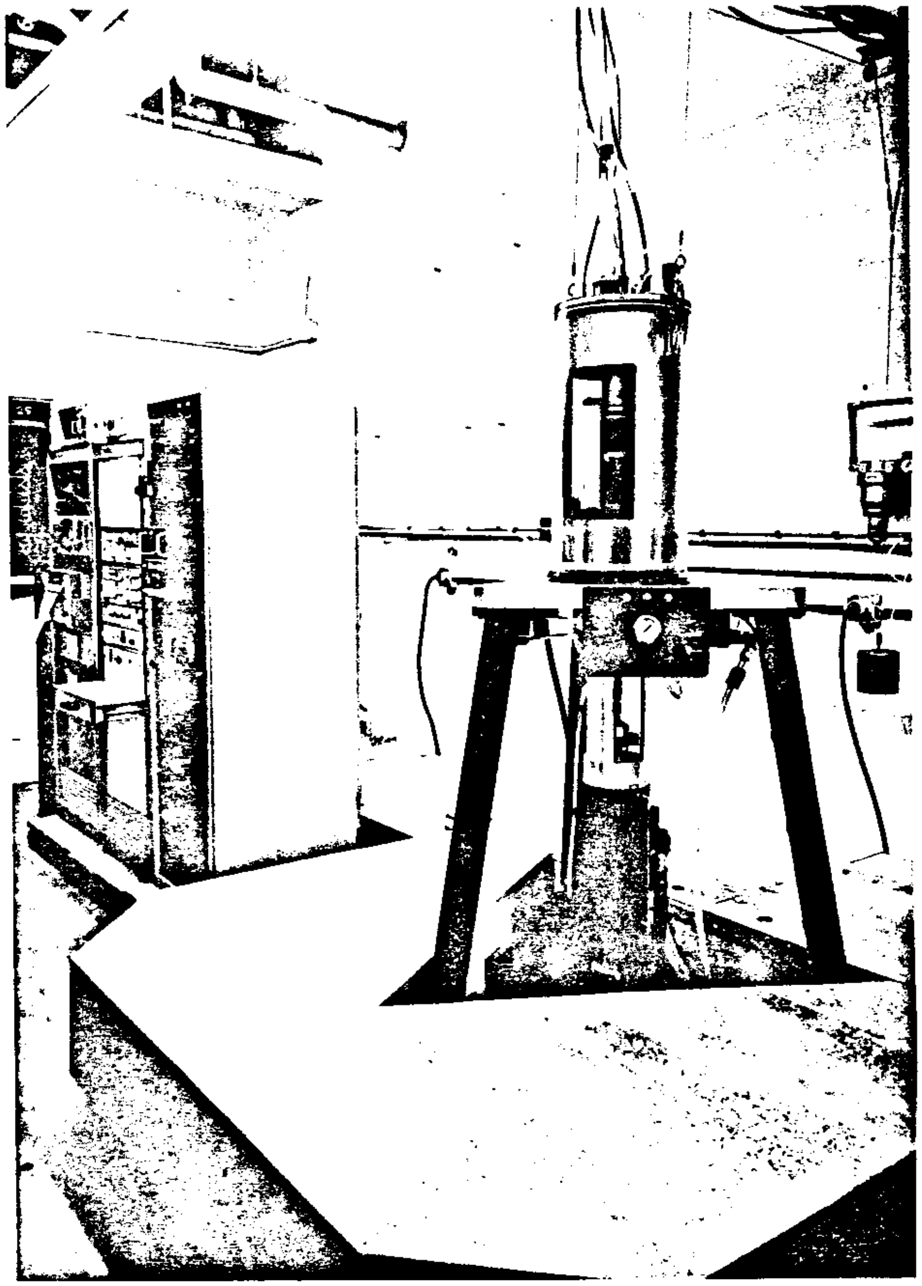


FIGURE 11 - DYNAMIC TRIAXIAL TESTING SYSTEM FOR CHARACTERIZATION OF PAVEMENT MATERIALS

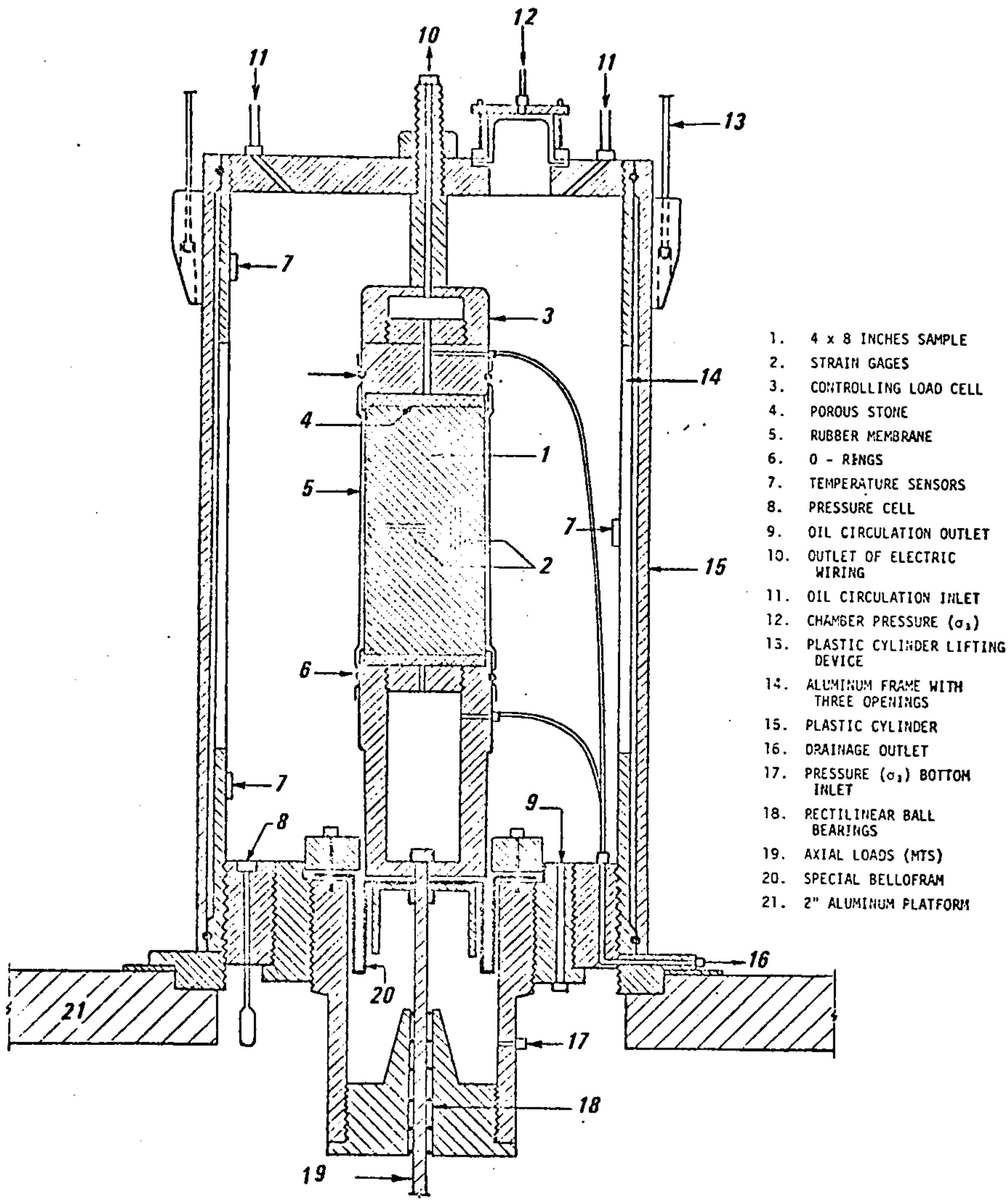


FIGURE 12 - CROSS SECTION OF TRIAXIAL CELL FOR DYNAMIC TESTING OF PAVEMENT MATERIALS

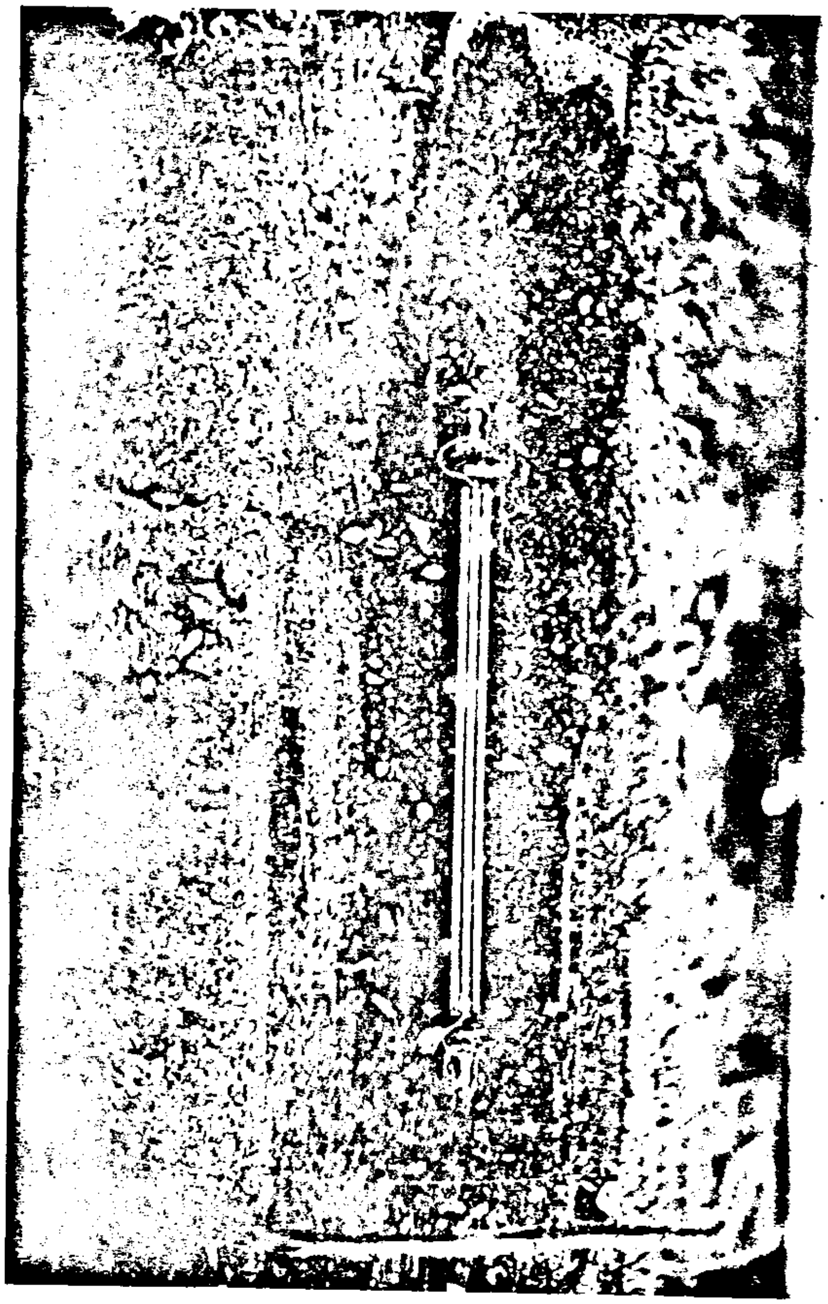


FIGURE 13 - CLOSE-UP OF 4" DIAM. x 8" HIGH ASPHALT CONCRETE SPECIMEN SHOWING 2" LONG STRAIN GAGE GLUED DIRECTLY TO SANDED SURFACE.

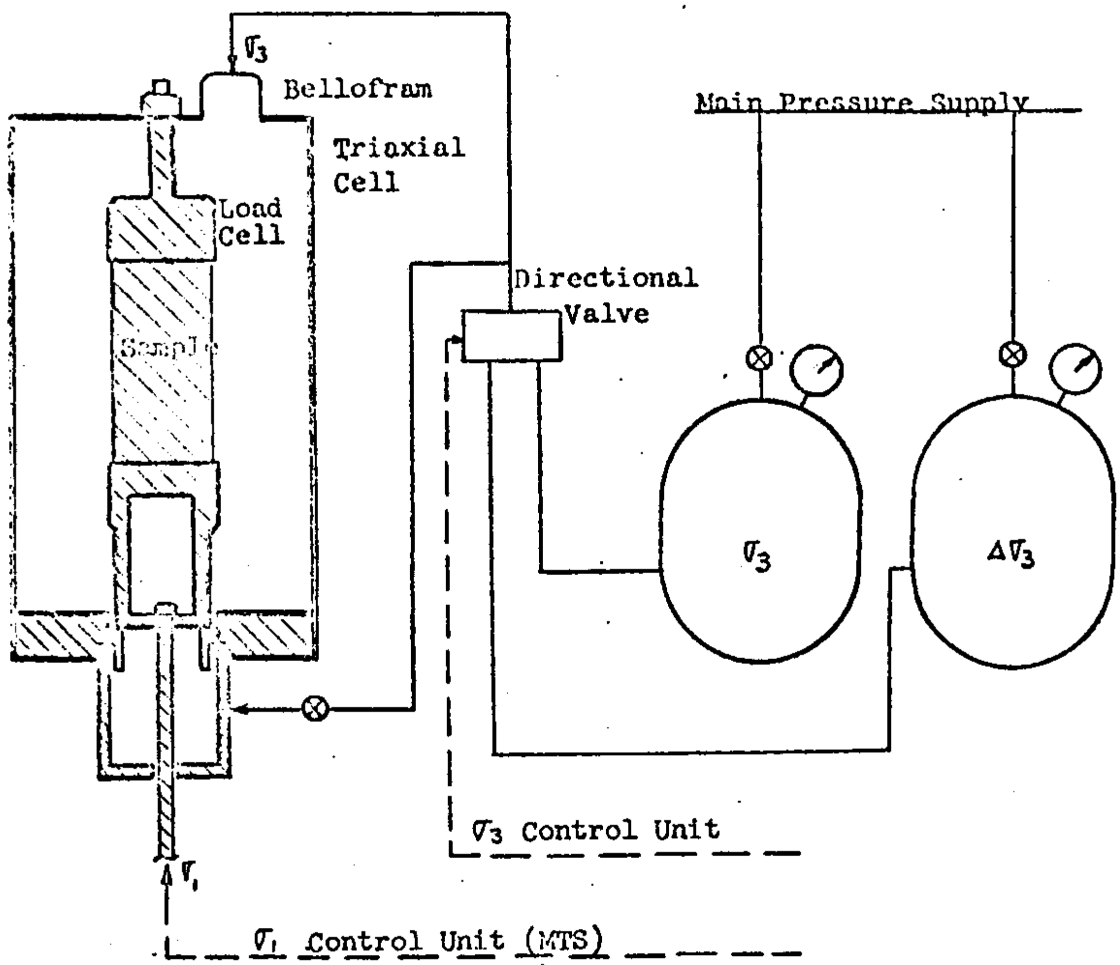


FIGURE 14 - A SYSTEM FOR APPLYING DYNAMIC LATERAL STRESS COINCIDENT WITH VERTICAL DYNAMIC STRESS

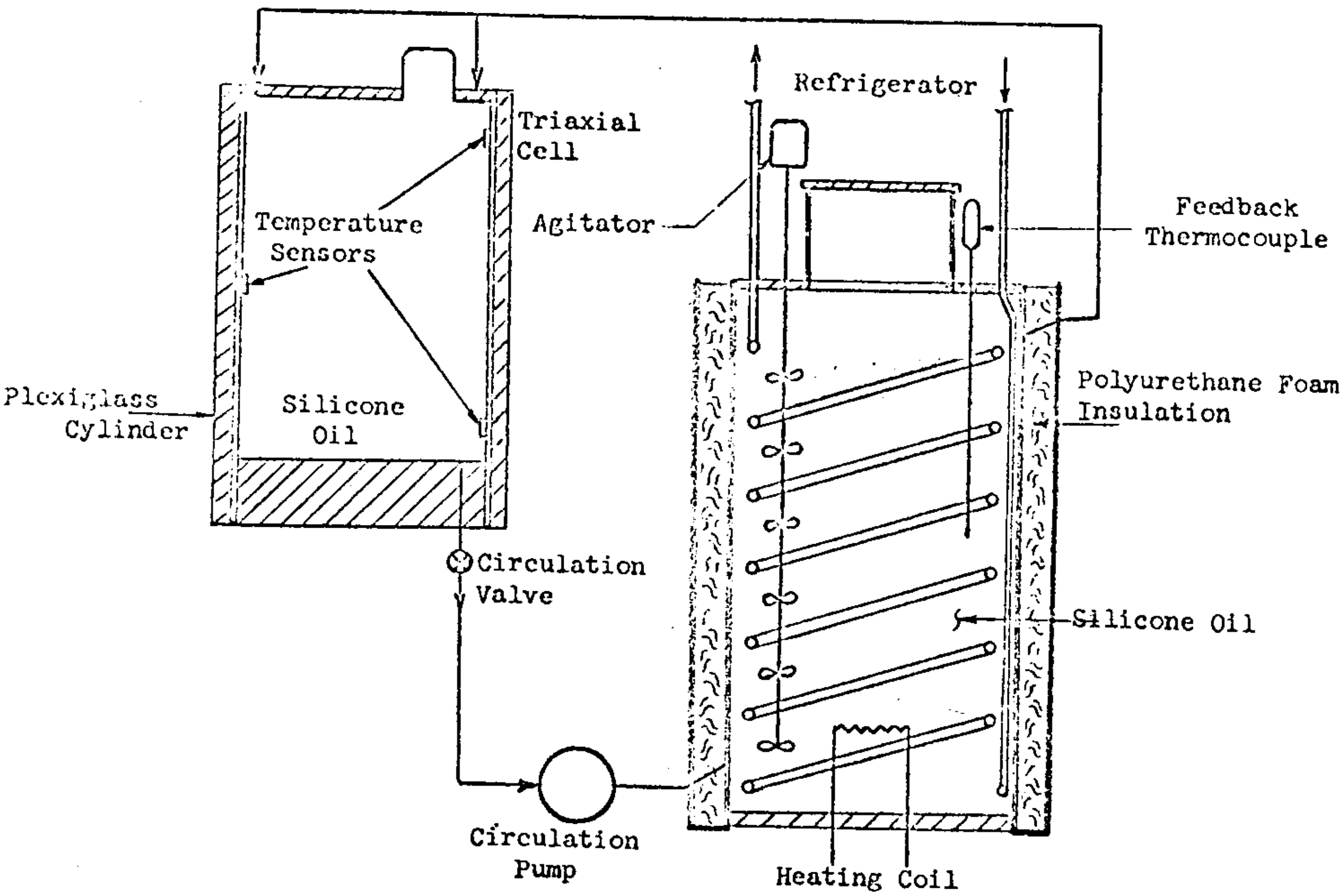


FIGURE 15 - A SYSTEM FOR PROVIDING CLOSED-LOOP TEMPERATURE CONTROL DURING DYNAMIC TESTS OF ASPHALT TREATED MATERIALS

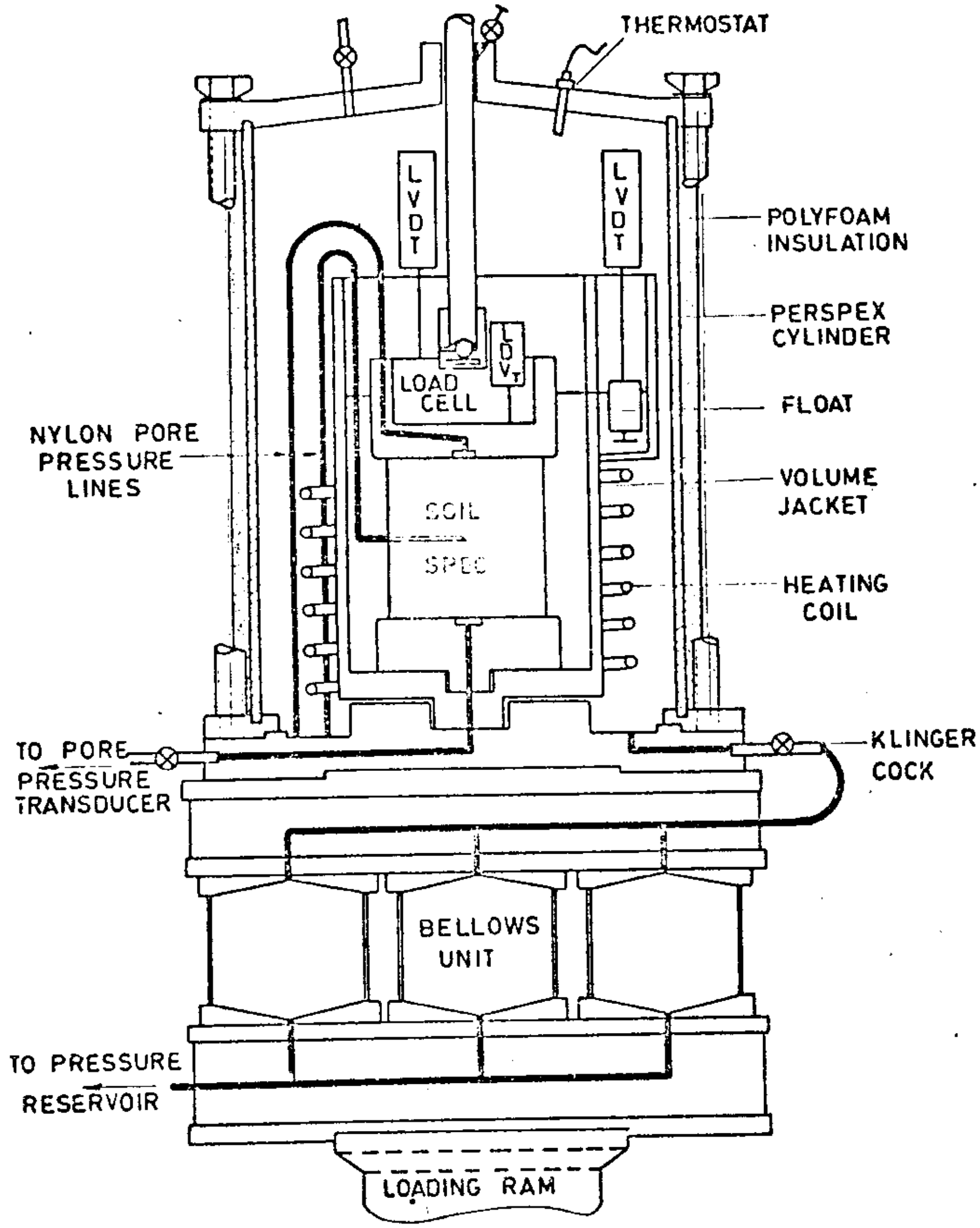


FIGURE 16 - ANOTHER SYSTEM FOR REPEATED-LOAD TESTING OF PAVEMENT MATERIALS (15)

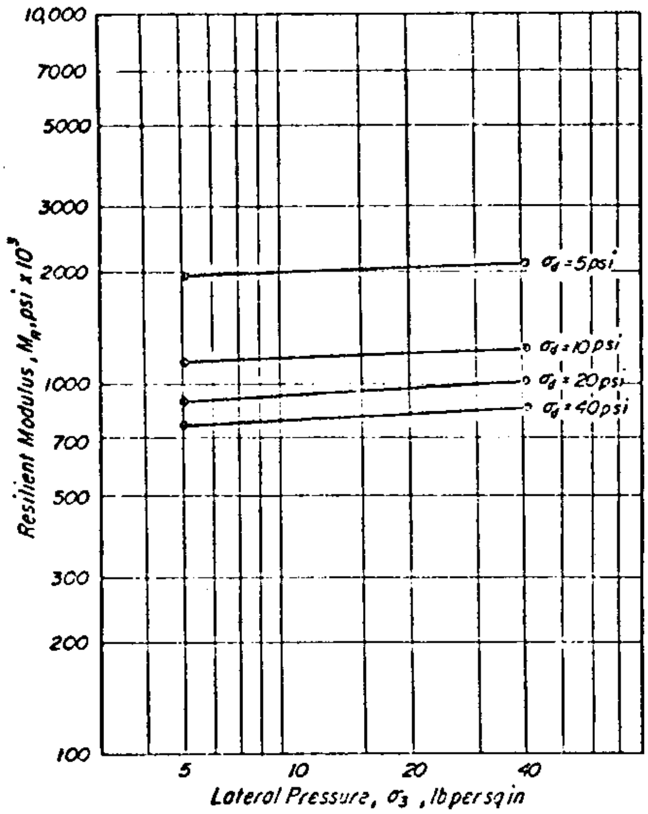


FIGURE 17 - TYPICAL RESILIENT MODULUS MEASUREMENTS FOR AN ASPHALT CONCRETE CYLINDER TESTED AT 0.1 SEC. LOAD DURATION (σ_d) AND 20 CPM. (17)

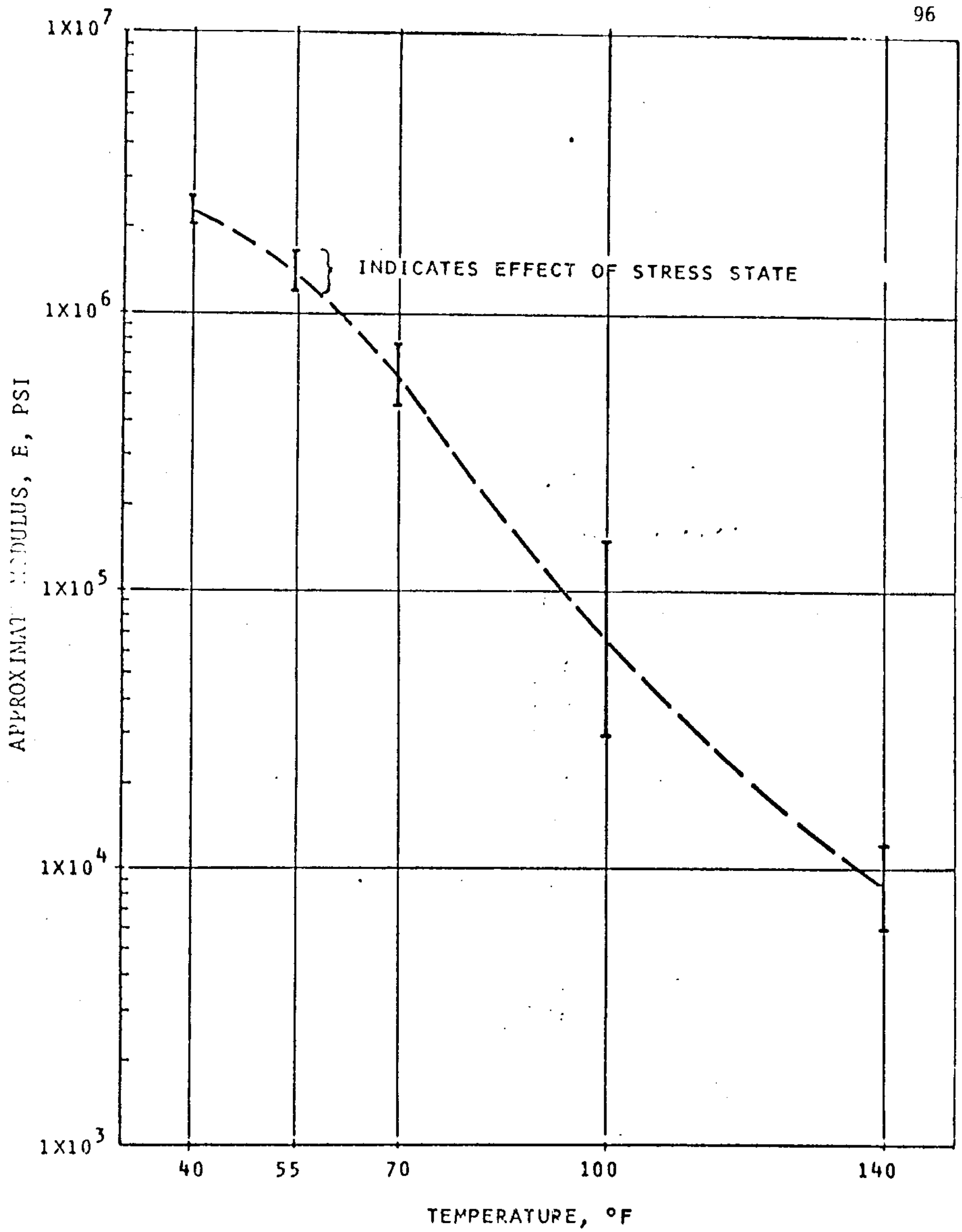


FIGURE 18a - TEMPERATURE DEPENDENCE OF RESILIENT MODULUS ASPHALT CONCRETE MIXTURES (43)

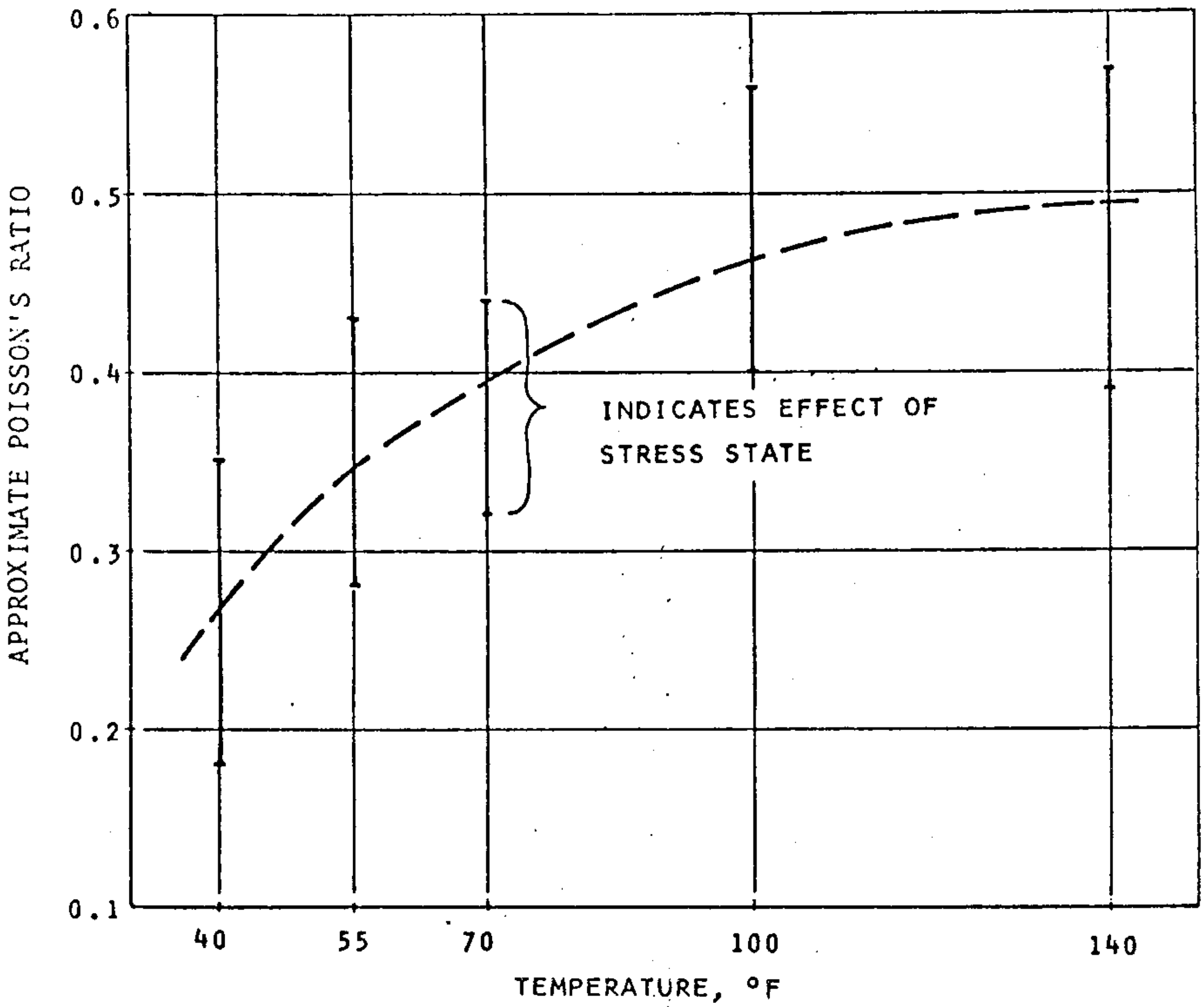


FIGURE 18b - TEMPERATURE DEPENDENCE OF POISSON'S RATIO FOR ASPHALT CONCRETE MIXTURES (43)

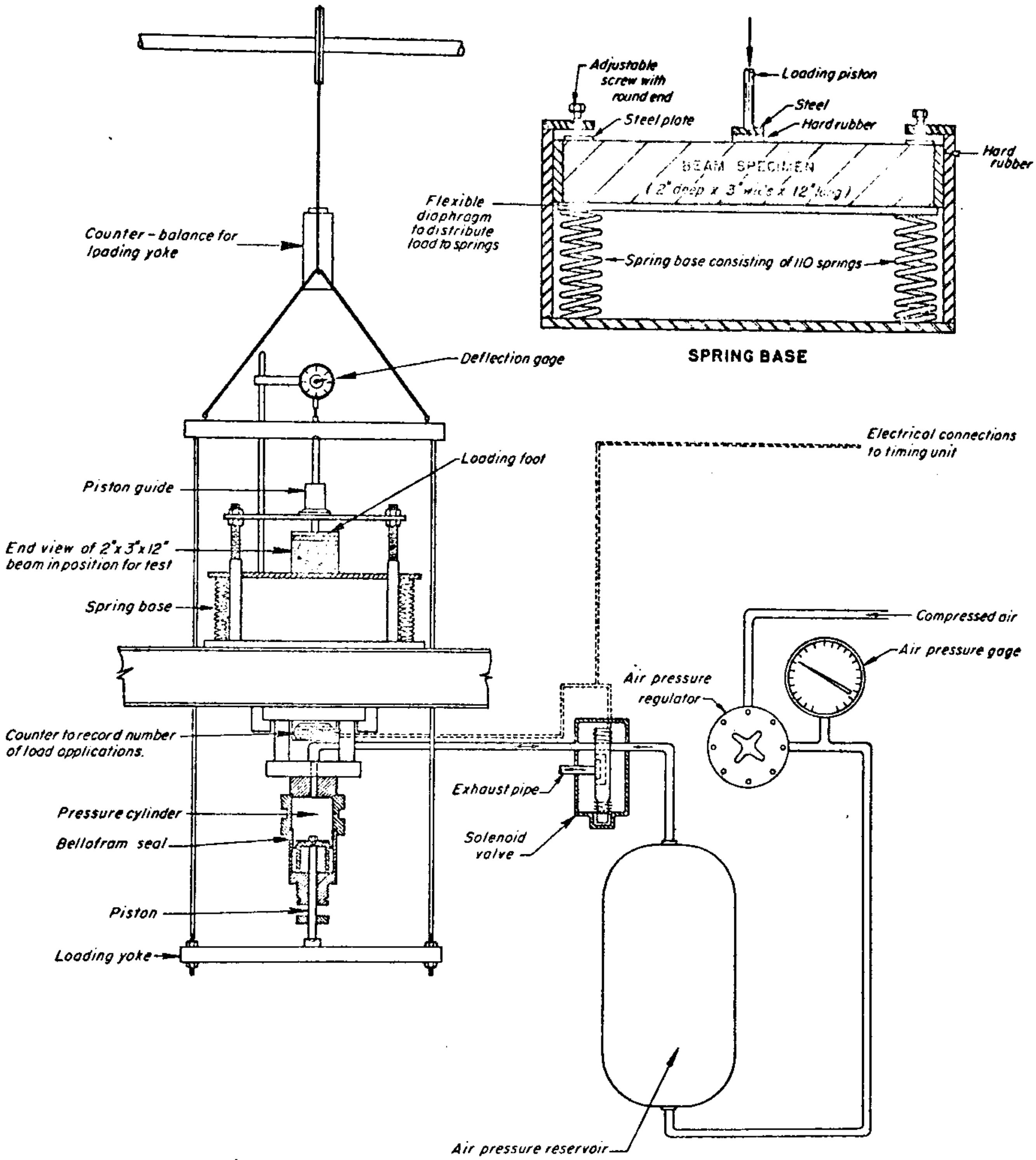
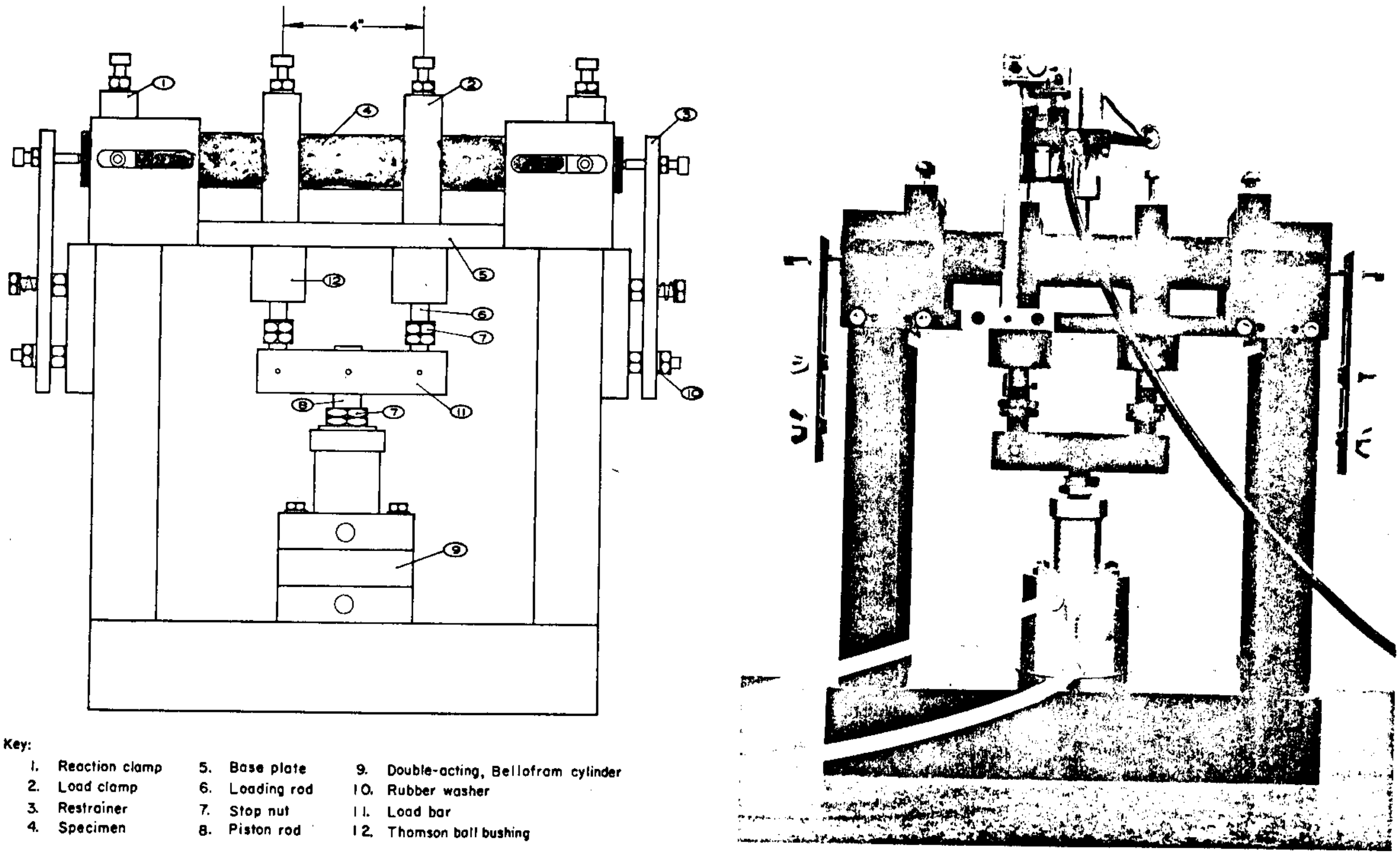


FIGURE 19 - EQUIPMENT FOR TESTING BEAMS IN REPEATED FLEXURE WHERE THE BASE IS A BED OF SPRINGS (19)



Key:

1. Reaction clamp	5. Base plate	9. Double-acting, Bellofram cylinder
2. Load clamp	6. Loading rod	10. Rubber washer
3. Restrainer	7. Stop nut	11. Load bar
4. Specimen	8. Piston rod	12. Thomson ball bushing

A

B

FIGURE 20 - LATER GENERATION OF THE REPEATED FLEXURE APPARATUS DEVELOPED AT THE UNIVERSITY OF CALIFORNIA, BERKELEY

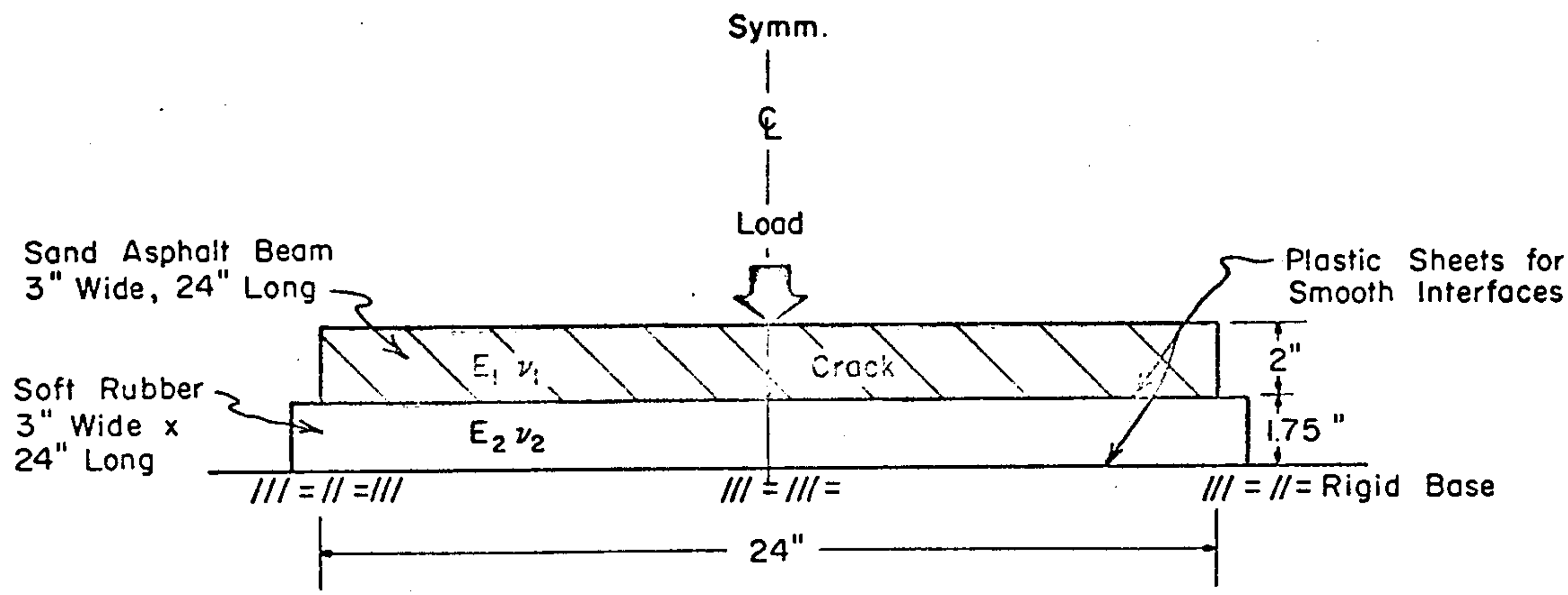


FIGURE 21 - TEST SPECIMEN ARRANGEMENT FOR BEAMS ON AN ELASTIC FOUNDATION (21)

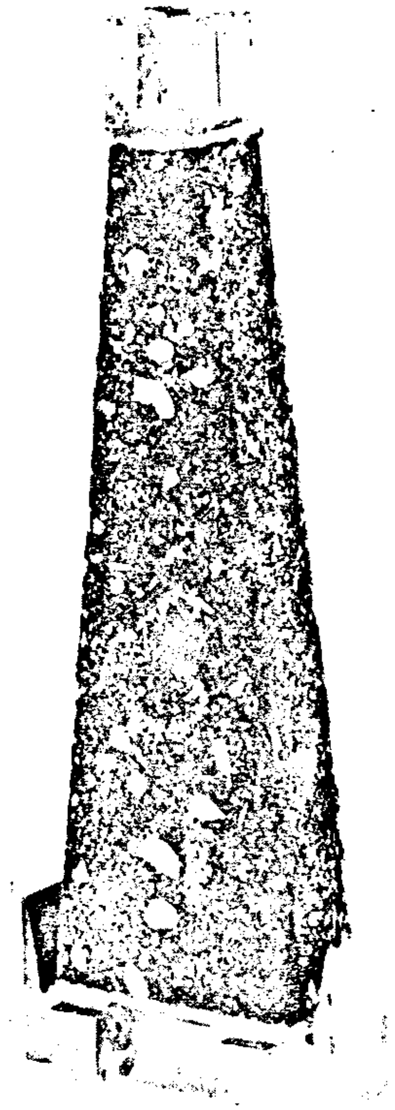


FIGURE 22 - SPECIAL TRAPEZOIDAL SPECIMEN FOR DYNAMIC STIFFNESS MEASUREMENTS IN CANTILEVER FLEXURE (17)

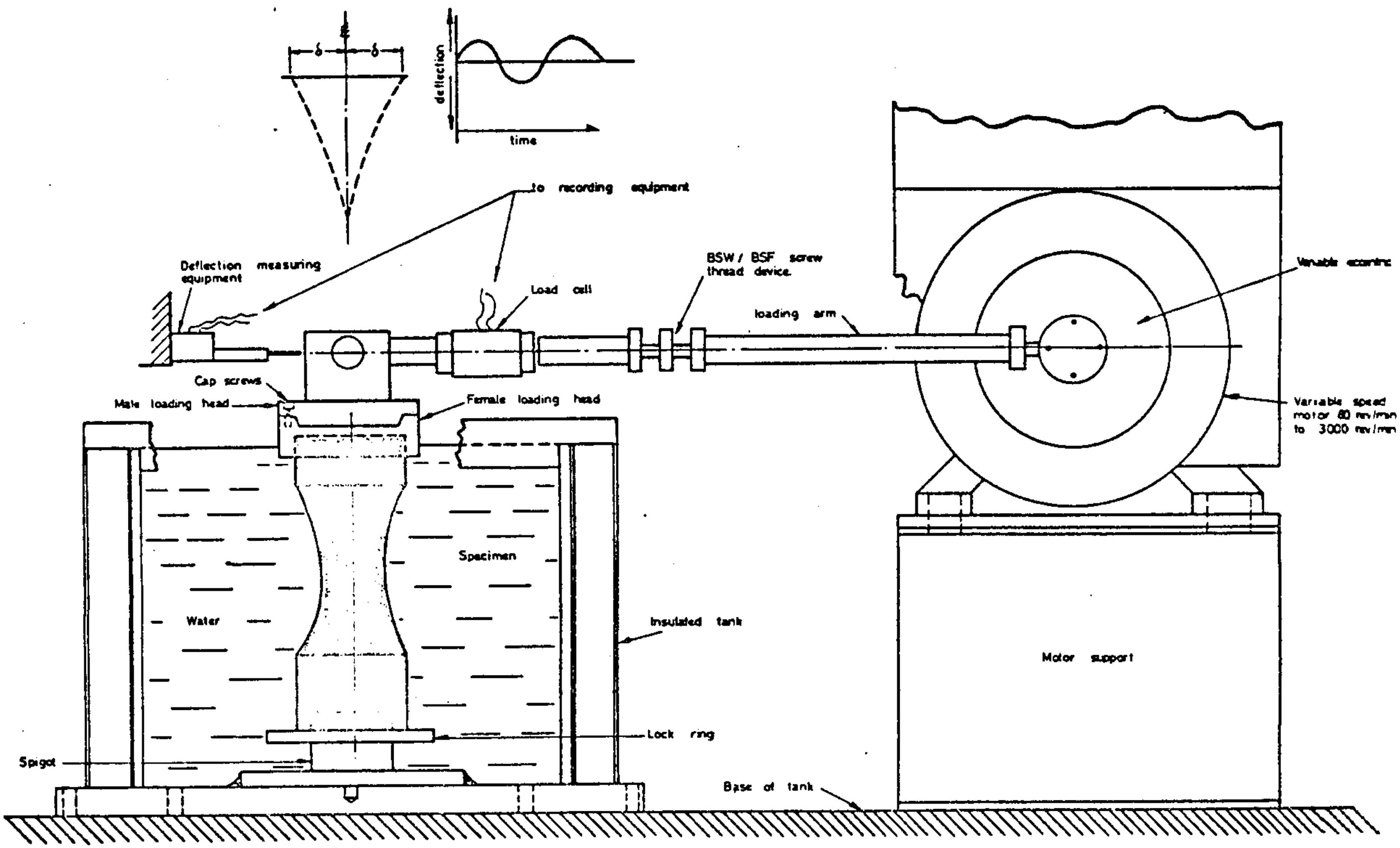


FIGURE 23 - ARRANGEMENT OF A MACHINE TO TEST SPECIALLY SHAPED CYLINDRICAL SPECIMENS IN DYNAMIC FLEXURE (23)

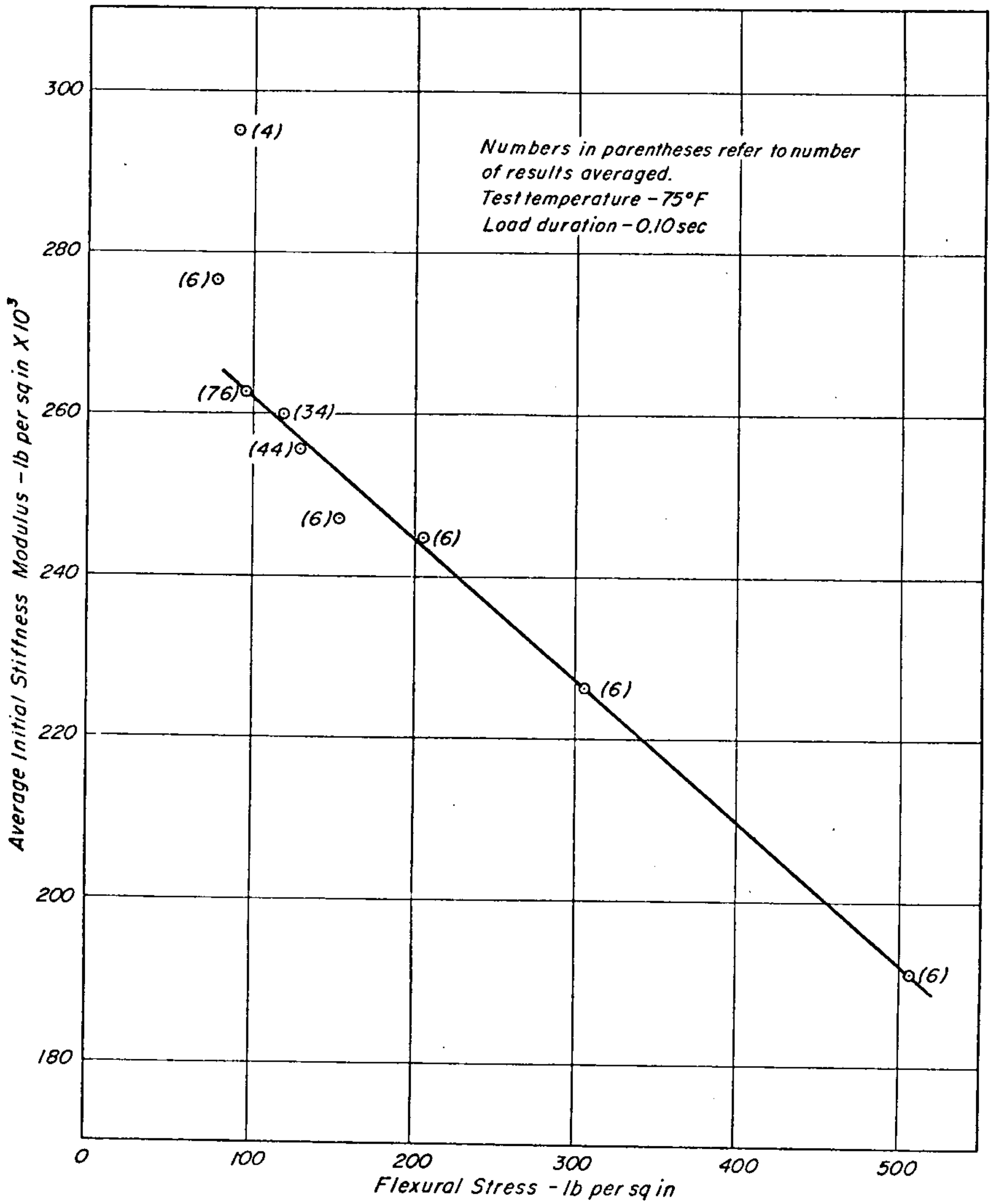


FIGURE 24 - RELATIONSHIP SHOWING STRESS DEPENDENCY OF STIFFNESS MODULUS IN REPEATED FLEXURE (24)

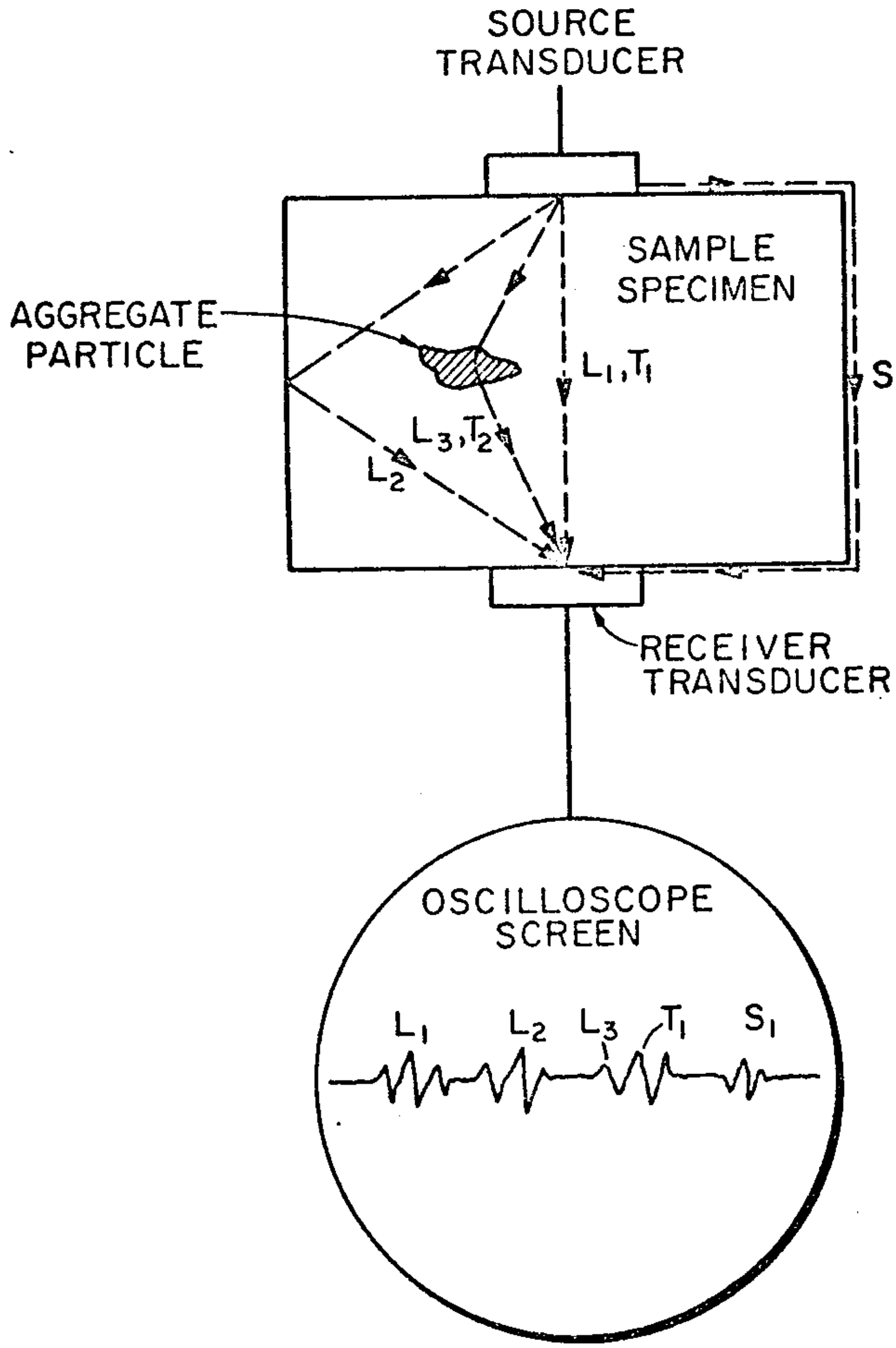


FIGURE 25 - ULTRASONIC TEST ARRANGEMENT SHOWING WAVE PATHS (29)

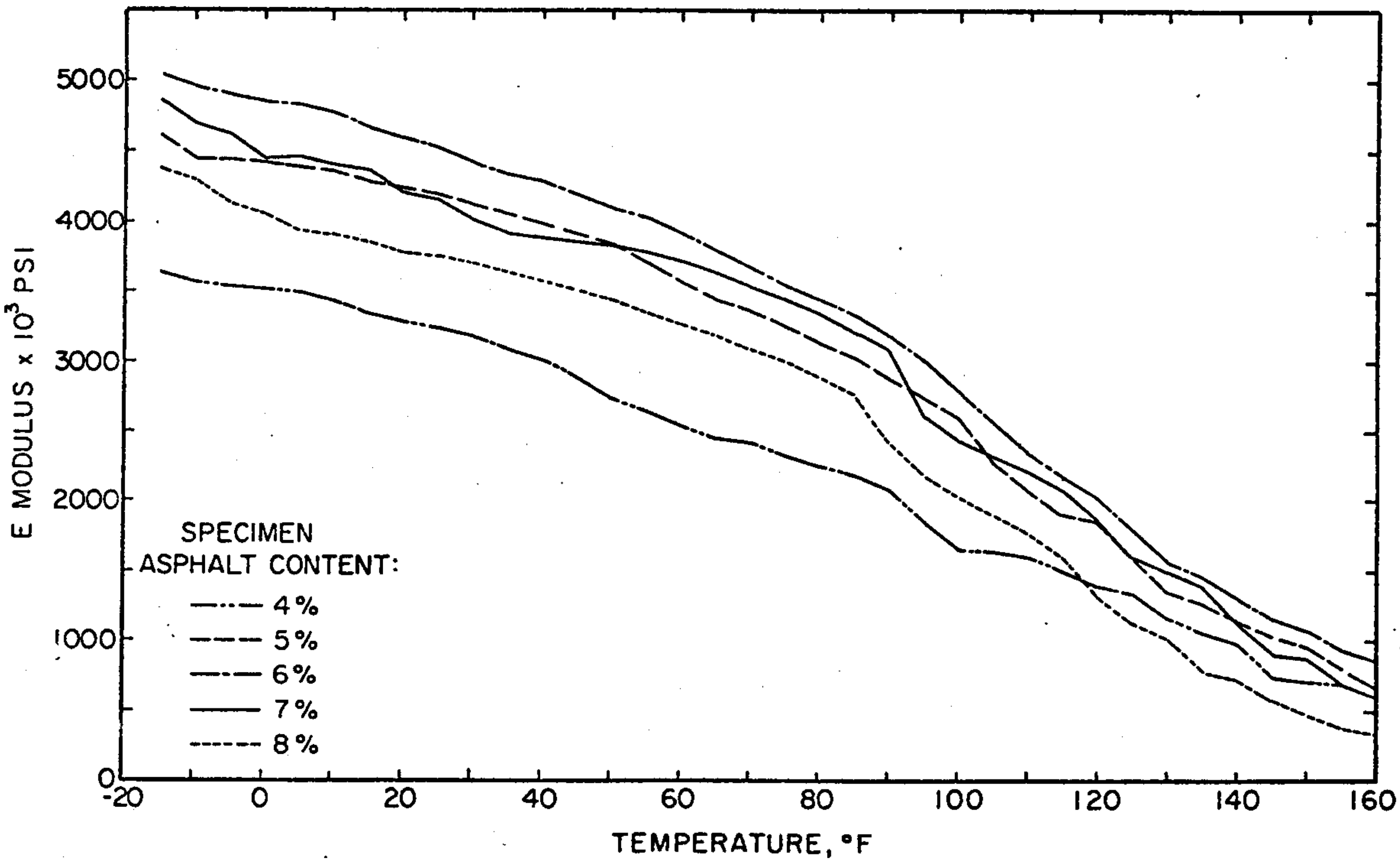


FIGURE 26 - EFFECT OF TEMPERATURE ON MODULUS FROM ULTRASONIC TESTS FOR A RANGE OF ASPHALT CONCRETE MIXTURES (29)

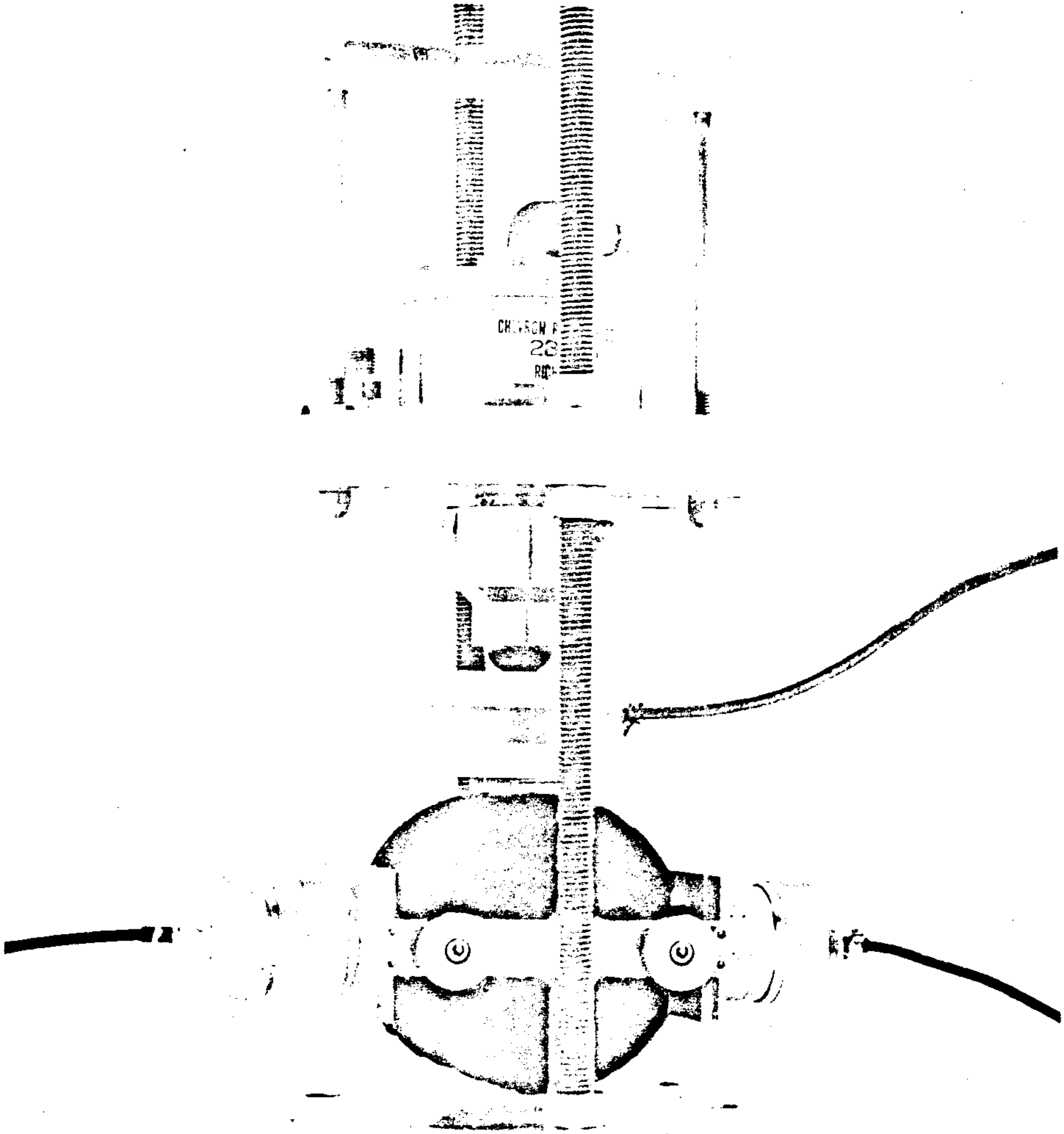


FIGURE 27 - SPECIAL TEST DEVICE FOR MEASURING THE RESILIENT MODULUS IN DIAMETRAL TENSION (26)

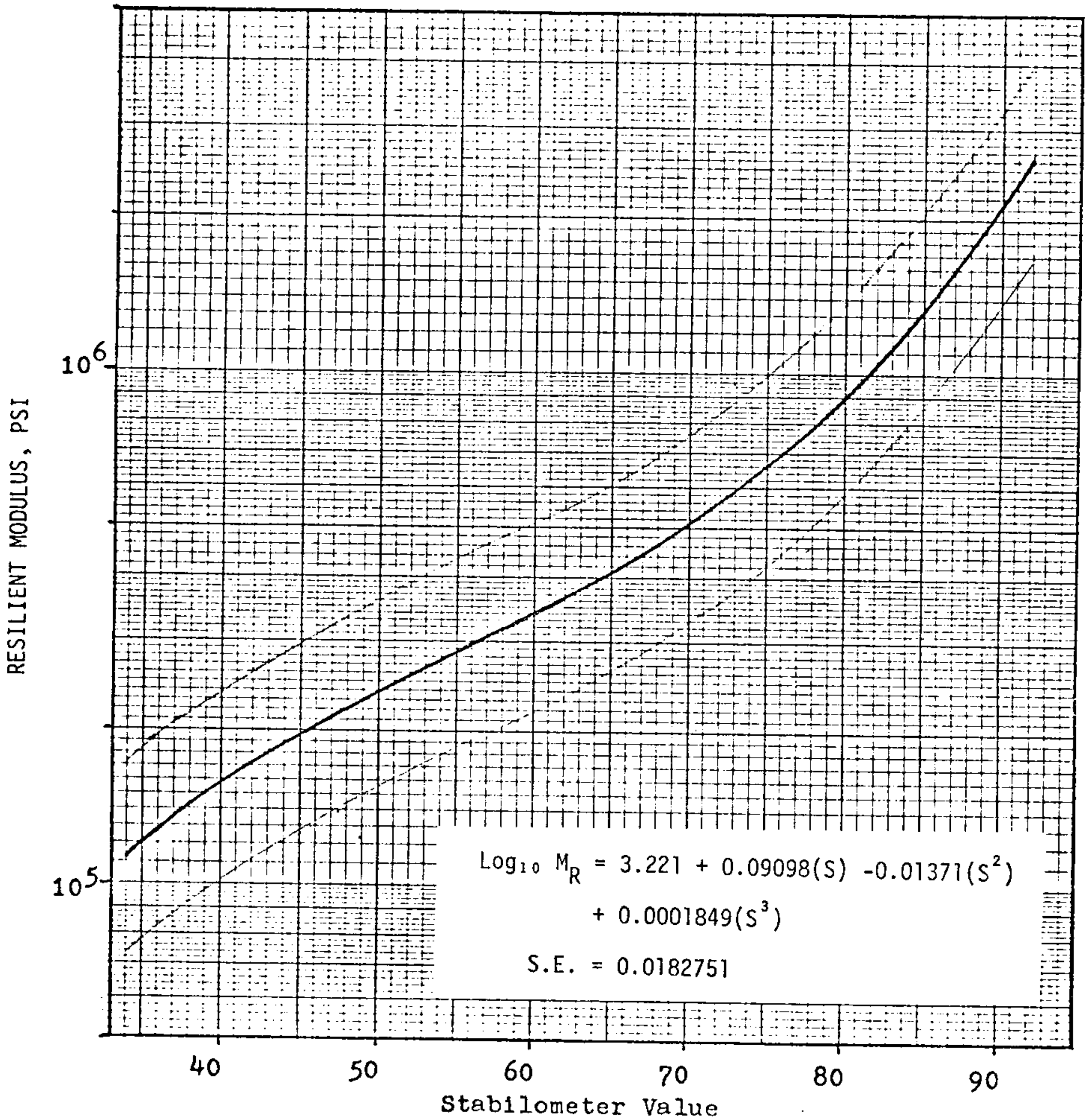


FIGURE 28 - CORRELATION WITH ERROR BOUNDS BETWEEN THE RESILIENT MODULUS AND S-VALUE FOR SEVERAL ASPHALT TREATED MIXTURES (16)

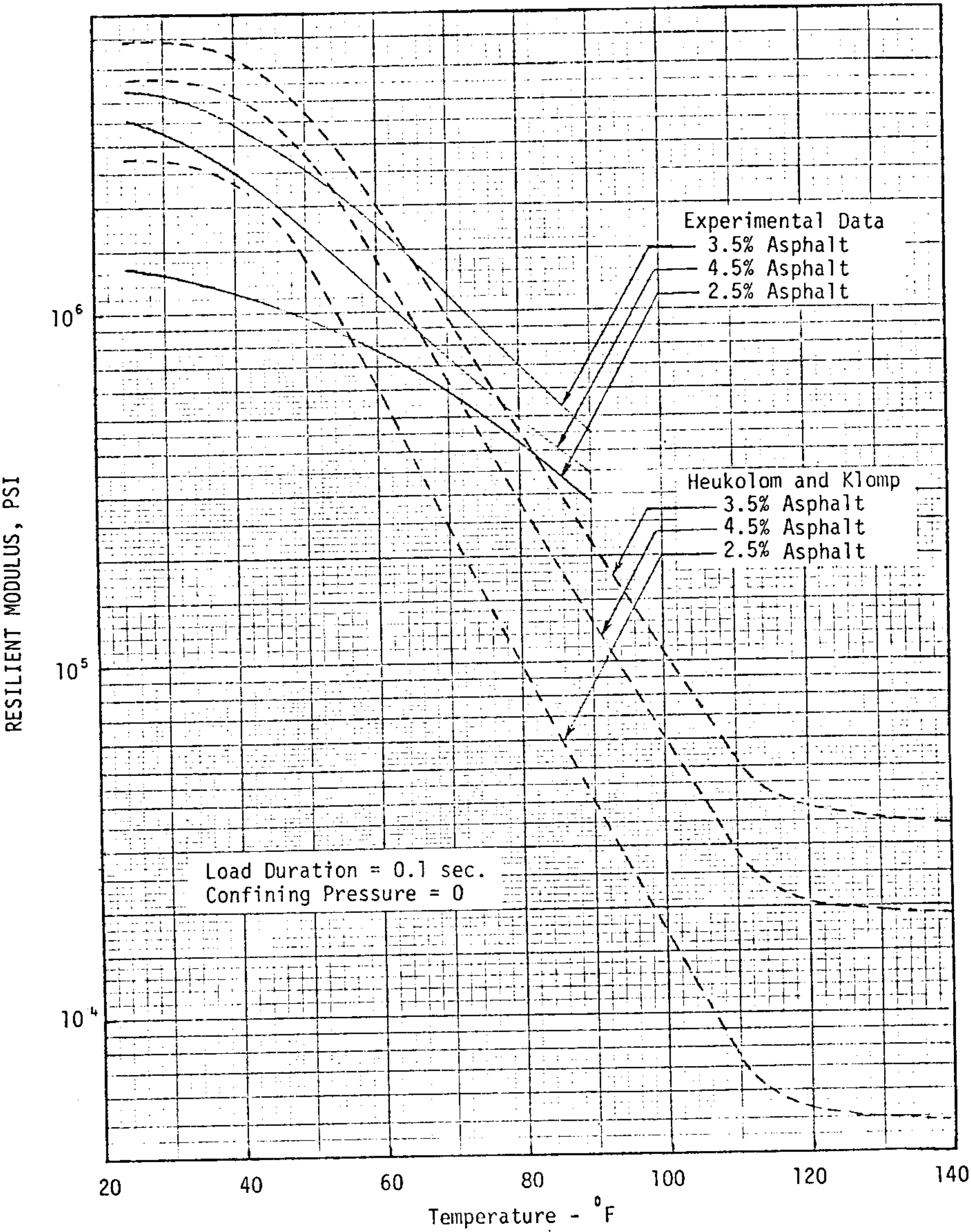


FIGURE 29 - RELATIONSHIP BETWEEN COMPUTED STIFFNESS AND MEASURED RESILIENT MODULUS FOR A RANGE OF BASE MIXES WITH LOW ASPHALT CONTENT

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ASPHALT TECHNOLOGY - CHEMICAL AND PHYSICAL CHARACTERISTICS

I. What is Asphalt

A. Where it comes from
B. Basic tests
   1) Consistancy test
   2) Safety test
   3) Contractural test

C. Importance of Asphalt Remaining in a Liquid State

II. Standard Types of Asphalts Available and Terminology Used

A. Asphalt Cements
B. Cutback Asphalts
C. Asphalt Emulsions
   1. Slurry seals
   2. Chip seals
D. Air Blown Asphalts
E. Mastics

III. Modified Asphalt Products

A. Epon-Asphalts for Bridge decking
B. Hot asphalt-rubber seal
C. Rubberized chip seals
D. Rubberized slurry seals

IV. Re-use of Asphalt Concrete and Use of In Place Materials

A. Misc. Base Material
B. Heater Remix
C. Complete Recycling-RMI process
D. Design Criteria
E. Emulsion Stabilization
F. Use of Lime

V. Problems in Asphalt Concrete

A. As caused by climatic conditions when placed
B. As caused by aggregates
C. As caused by asphalt type and consistancy
D. As caused by climatic conditions during use life.

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* Notes prepared by Robert Dunning

RHEOLOGIC BEHAVIOR OF ASPHALTS AND ASPHALT MIXTURES

Introduction

Investigations of the rheological or stiffness properties of asphalt and asphaltic concrete have shown that they are time-of-loading and temperature dependent and can be expected to act elastically for specific conditions. For example, as long as the temperature and rate of loading do not vary markedly, a mixture of asphaltic concrete will act elastically up to approximately 0.1 percent strain. Thus, it is possible to analyze asphaltic mixtures according to the theory of elasticity for a given situation as represented by the modulus of elasticity or stiffness modulus. Methods have also been developed, or carried over from related technical disciplines, which make it possible to readily predict the stiffness modulus over a wide range of temperatures and times of loading from a comparatively limited number of tests. There is not complete unanimity as regards the applicability of treating asphaltic mixtures as elastic or linear-viscoelastic materials; however, there appears to be a strong consensus that this approach should be pursued as far as possible before attempting to deal with this material by more complicated methods.

Background

Rheology - study of flow of materials - Thus interested in relation between stress and strain as a function of time. Elasticity can be considered as a special case in rheology where the stress vs. strain characteristic is time independent.

For an elastic material

$\sigma = E \cdot \epsilon$ and $\tau = G \cdot \gamma$

where: E - elastic modulus in tension or compression and σ and ϵ are corresponding stress and strain.

and G - elastic modulus in shear and τ and γ are corresponding stress and strain.

For a viscous material

$\sigma = \lambda \cdot \frac{d\epsilon}{dt}$ and $\tau = \eta \cdot \frac{d\gamma}{dt}$

where: λ = viscous traction in tension or compression

$\frac{d\epsilon}{dt}$ = rate of axial strain

η = viscosity in shear

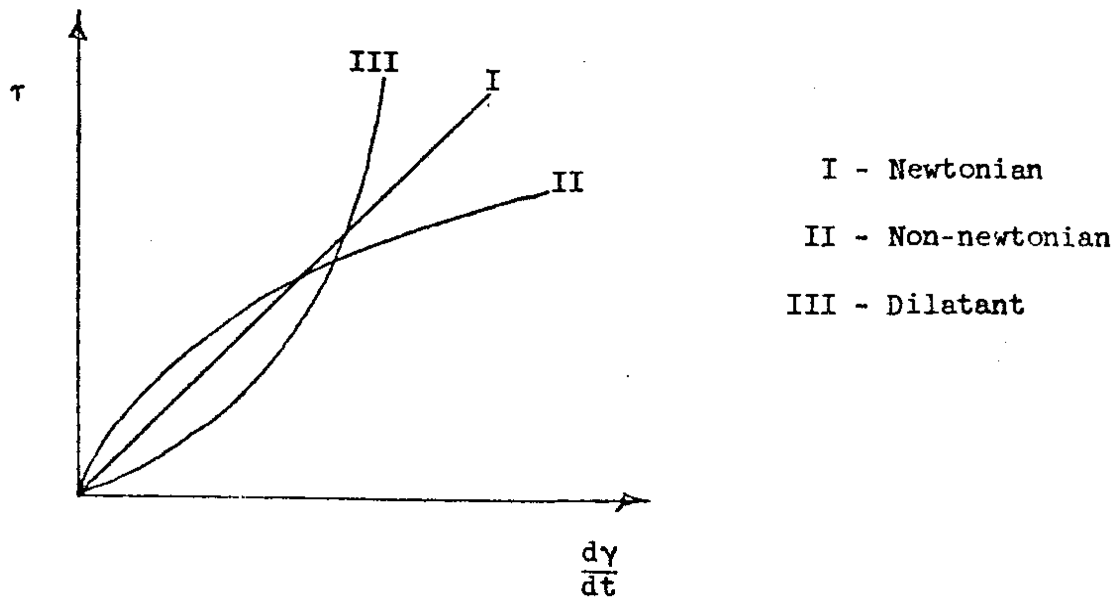
$\frac{d\gamma}{dt}$ = rate of shear strain

For an isotropic elastic solid = $G = \frac{E}{2(1 + \mu)}$

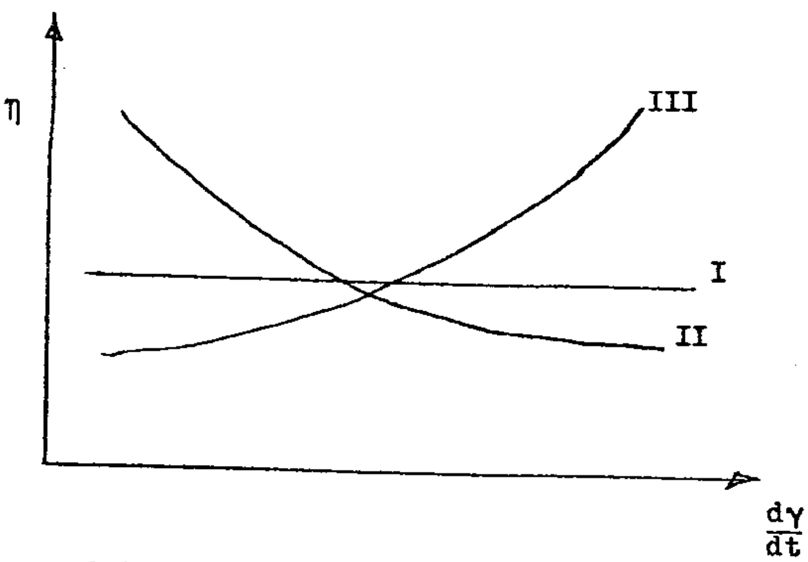
μ = Poisson's ratio

For an incompressible fluid: $\lambda = 3\eta$

In the case of viscous materials at least three types of flow possible. These are illustrated in accompanying sketch.



Considering viscosity as a function of $\frac{d\gamma}{dt}$, viscosity will vary for the three types of behavior as follows:

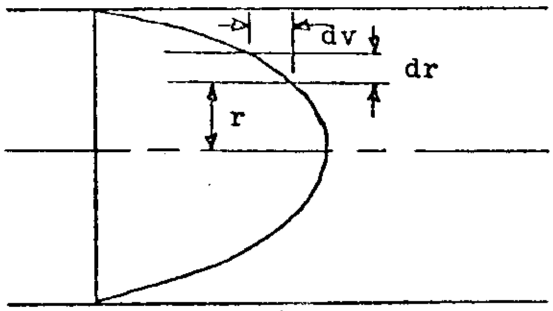


Viscous behavior can be determined in a number of different ways. In case of asphalts, common instruments now is use:

1. Capillary viscometer
2. Falling plunger viscometer
3. Sliding plate microviscometer

Viscosity determined from results of capillary viscometer with aid of Poiseuille's law:

Consider fluid flowing in capillary under laminar flow conditions;



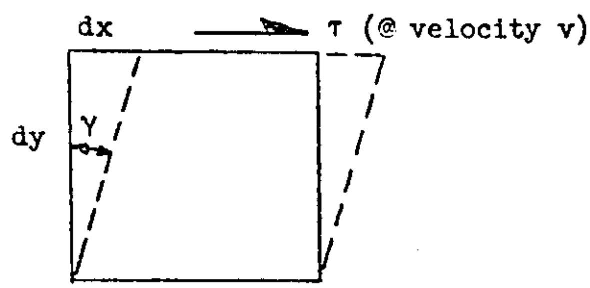
frictinal resistance between layers of fluid proportional to velocity gradient dv/dr

and $q = \frac{1}{8\pi\nu} \cdot i \cdot a^2$

where $\nu = \frac{\eta}{\text{density}}$

Thus: $\nu = \frac{1}{8\pi q} \cdot i \cdot a^2 = \underset{\text{(constant)}}{C} \cdot t \quad \left\{ \begin{array}{l} \text{for a} \\ \text{particular} \\ \text{capillary} \end{array} \right\}$

Previously had indicated that $\tau \approx d\gamma/dt$; also approximately equal to dv/dr which is used in many viscometer measurements. This can be shown as follows:



Consider an element deformed in shear

$$\frac{dv}{dy} \text{ (same as } \frac{dv}{dr} \text{ above)} = \frac{d\,\frac{dx}{dt}}{dy}$$

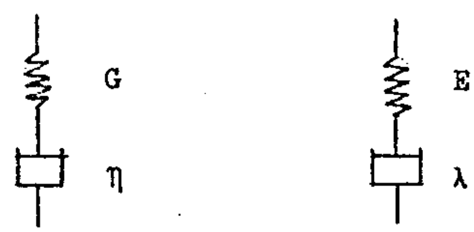
$$= \frac{d\,\frac{dx}{dy}}{dt} = \frac{d \tan\gamma}{dt} \approx \frac{d\gamma}{dt}$$

Can combine types of behavior described above to produce viscoelastic behavior. One approach is to consider material in terms of Hookean (linear) springs and Newtonian dashpots - referred to as linear viscoelastic behavior.

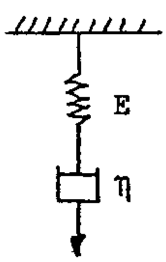
This is purely a mechanistic approach, gives measure of performance at <u>phenomonological</u> level not at <u>microanalytical</u> level.

Can be useful approach in considering behavior of materials where time rate of loading is a factor. In addition, should be emphasized that approach is limited to small deformations and failure is not considered.

Various ways of representing simple linear behavior; consider first <u>Maxwell</u> body:



For stress applied to the system:



Note: η rather than λ used in succeeding discussion

$$\epsilon_{total} = \epsilon_{spring} + \epsilon_{dashpot} \quad . . . (1)$$

$$\text{and } \sigma_{total} = \sigma_{spring} = \sigma_{dashpot}$$

differentiating eq. (1) w.r.t. time

$$\frac{d\epsilon}{dt} = \frac{d\epsilon_s}{dt} + \frac{d\epsilon_d}{dt}$$

$$\frac{d\epsilon}{dt} = \frac{1}{E} \cdot \frac{d\sigma}{dt} + \frac{\sigma}{\eta} \quad . . . . (2a)$$

This equation can also be expressed in operator form.

Let $p = \frac{d}{dt}$

$$p\epsilon = \left(\frac{p}{E} + \frac{1}{\eta} \right) \sigma \qquad (2b)$$

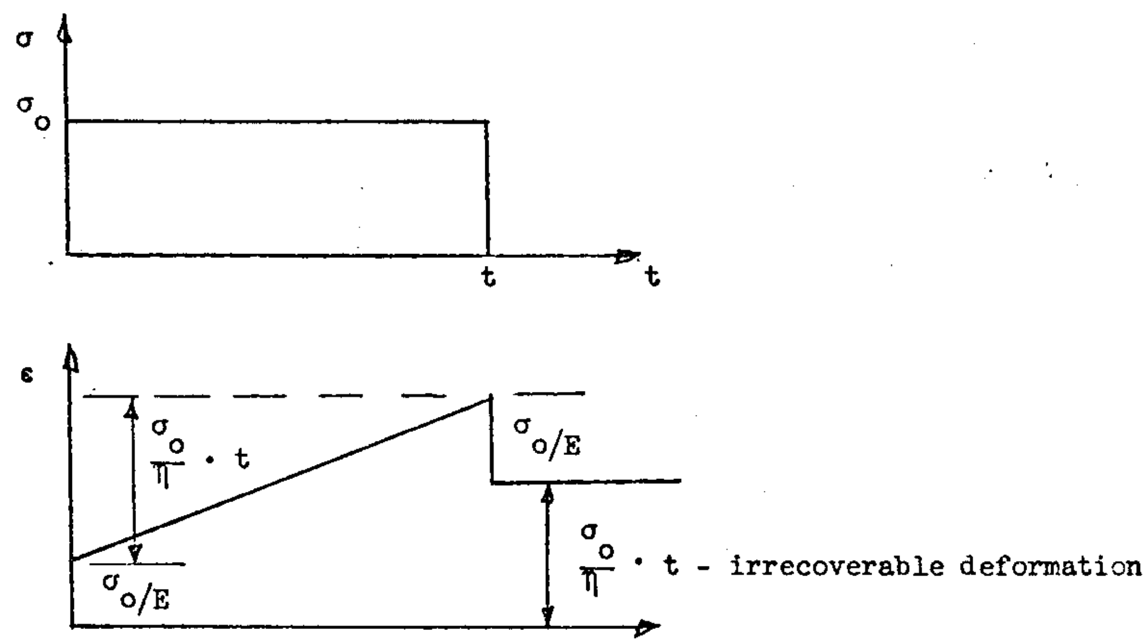
Consider solution of eq. (2) for number of conditions:

1. Creep

for: $\sigma = \sigma_o$ = constant

$$\epsilon = \sigma_o \left(\frac{1}{E} + \frac{t}{\eta} \right) \qquad (3)$$

This can be visualized as follows:

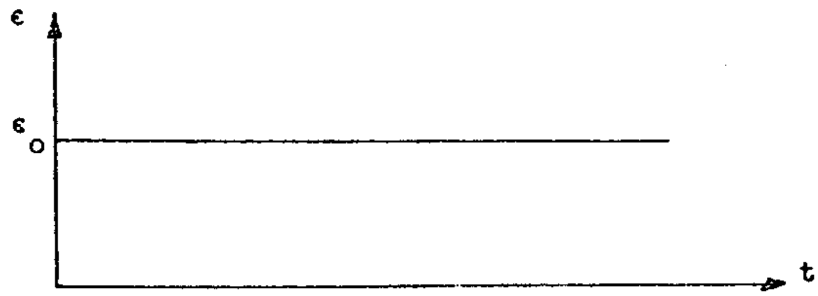


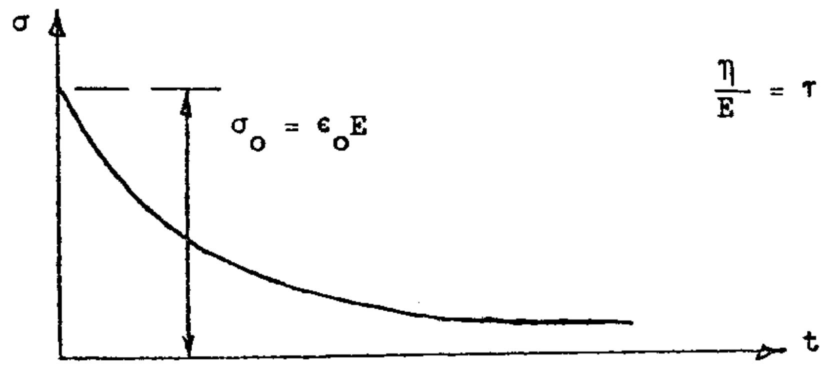
2. Relaxation

for: $\epsilon = \epsilon_o$ = constant

$$\sigma = \sigma_o e^{-\frac{E}{\eta} t} = \sigma_o e^{-\frac{t}{\tau}} \qquad (4)$$

where $\tau = \frac{\eta}{E}$ = relaxation time



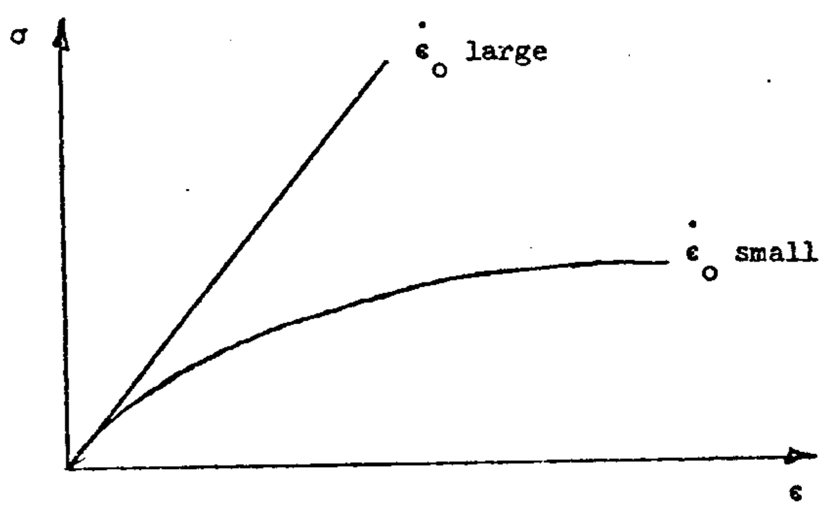


$\frac{\eta}{E} = \tau$ represents time required for initial stress to decrease to value σ_o/e

3. <u>Constant Rate-of-Strain</u>

$$\frac{d\epsilon}{dt} = \text{constant} = \dot{\epsilon}_o$$

$$\sigma = \tau E\dot{\epsilon}_o \left[1 - e^{-t/\tau} \right] \quad \ldots\ldots\ldots\ldots\ldots \quad (5)$$



4. Dynamic Vibratory Test

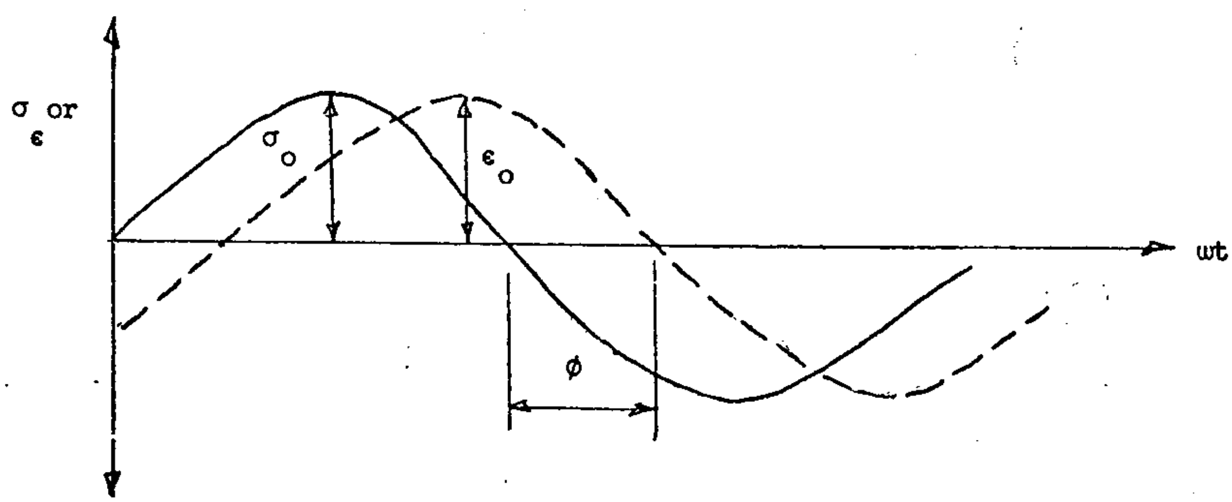
For response to simple harmonic stress

$$\sigma = \sigma_o \cos \omega t \qquad \{(\ \omega = \text{angular frequency}\ )\}$$

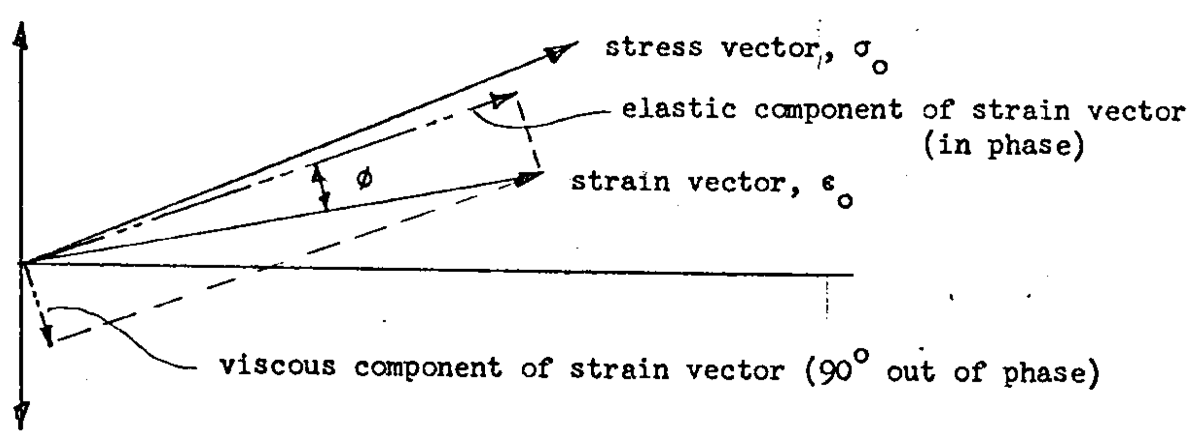
$$\epsilon = \epsilon_o \cdot \sin(\omega t - \phi) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

where $\epsilon_o = \frac{\sigma_o}{E\omega\tau} (1 + \omega^2\tau^2)^{\frac{1}{2}}$

and $\phi = \tan^{-1} \cdot \frac{1}{\omega\tau}$ (termed phase angle)



Can also use rotating vector representation to visualize:



Can also consider complex modulus, E^*

$$\left|E^*\right| = \frac{\sigma_o}{\epsilon_o} = \frac{E\omega\tau}{(1 + \omega^2\tau^2)^{\frac{1}{2}}} \quad \ldots\ldots\ldots\ldots\ldots (7)$$

real part of complex modulus or storage modulus

$$E' = \left|E^*\right| \cos\phi = \frac{E\omega^2\tau^2}{1 + \omega^2\tau^2} \quad \ldots\ldots\ldots\ldots (8)$$

(stress divided by component of strain in phase with stress)

loss modulus:

$$E'' = \left|E^*\right| \sin\phi = \frac{E\omega\tau}{1 + \omega^2\tau^2} \quad \ldots\ldots\ldots\ldots (9)$$

(stress divided by component of strain 90° out of phase with stress)

Another convenient way of representing responses is through use of operators (already shown on p. 5)

$$p\epsilon = \left(\frac{p}{E} + \frac{1}{\eta} \right) \sigma$$

$$pE\epsilon = \left(p + \frac{1}{\tau} \right) \sigma$$

or $p\sigma = QE\epsilon$

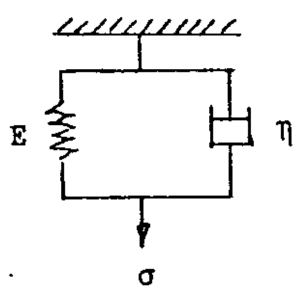
and $\sigma = \frac{Q}{p} E \cdot \epsilon = \overline{E} \cdot \epsilon$

(note similarity to elasticity)

(analogous to Van der Poel's Stiffness)

Above equations cover several types of responses that will be encountered.

Consider next Kelvin or Voight element:



$$\sigma_{total} = \sigma_{spring} + \sigma_{dashpot} \quad \ldots\ldots\ldots\ldots (10)$$

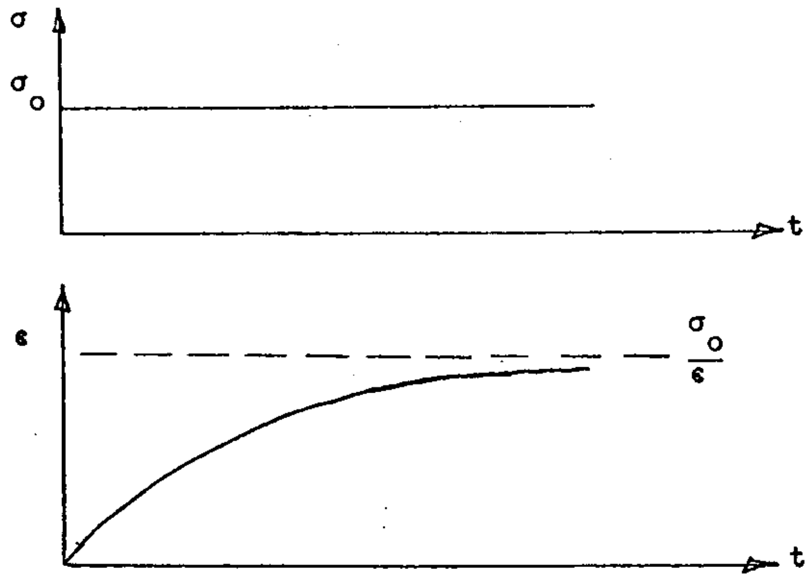
$$\epsilon_{spring} = \epsilon_{dashpot} = \epsilon_{total}$$

Thus $\sigma = E \cdot \epsilon + \eta \frac{d\epsilon}{dt}$ $\quad \ldots\ldots\ldots\ldots (11)$

For $\sigma = \sigma_o$ (creep); solution of equation (11) is:

$$\epsilon = \frac{\sigma_o}{E} \left(1 - e^{\,t/\tau} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (12)$$

where: $\tau = \frac{\eta}{E}$ and in this case is called the retardation time.



example of retarded elastic behavior

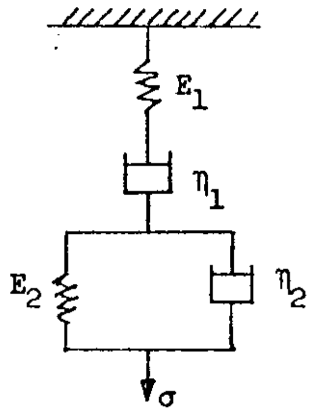
Can also write equation (12) as:

$$\epsilon = J\sigma_o \left(1 - e^{-\,t/\tau} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (13)$$

where: J = compliance and in this case (and only in this case) equals $\frac{1}{E}$.

Generally, engineering materials cannot be represented by either of these simple elements. Must consider various combinations of simple elements in series and parallel arrays to simulate behavior of real materials.

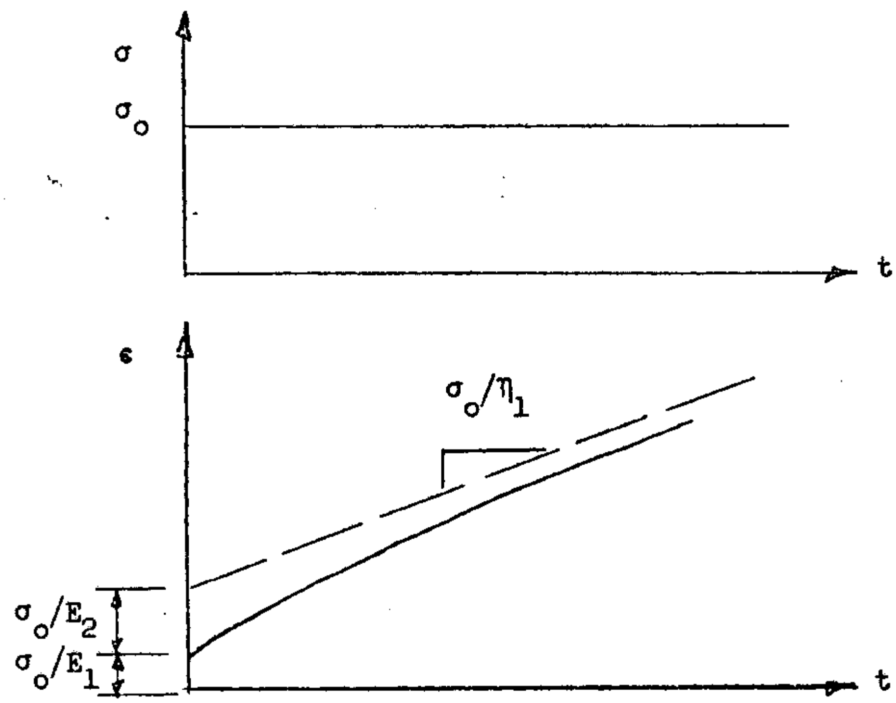
One possible combination using both the Maxwell and Kelvin elements is shown in the accompanying sketch, termed a four element (or Burgers) model.



Response to creep load $\sigma = \sigma_o$

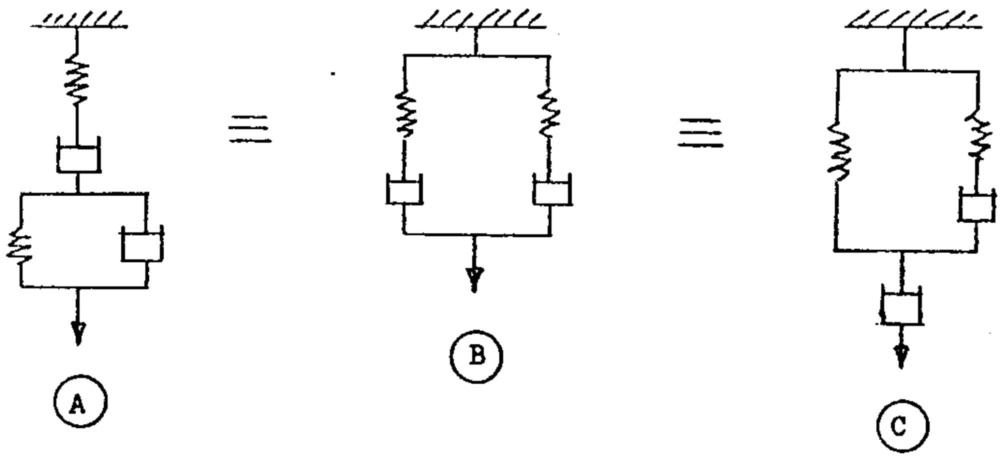
$$\epsilon = \frac{\sigma_o}{E_1} + \frac{\sigma_o}{E_2}\left[1 - e^{-\frac{t}{\tau_2}}\right] + \frac{\sigma_o}{\eta_1} \cdot t \quad . . . \quad (14)$$

Can be visualized:

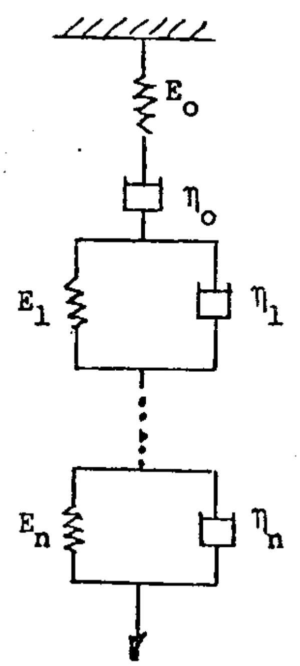


Should be noted at this point that number of arrangements of elements can be obtained which are equivalent to each other. The E's and the η's will have different values but the overall behavior under time-dependent loading will be the same. Which configuration is chosen depends on which will permit easiest solution of differential equation for particular problem.

Can note the following:



This type of approach can be extended to include finite or infinite number of elements. With the advent of the computer no advantage in considering finite number of elements; much better to work with generalized distributions. Models help to visualize generalized behavior, however, since generalized expressions relating stress, strain and time follow from simpler representations. For creep, convenient to consider generalized Kelvin model with a spring and dashpot in series.



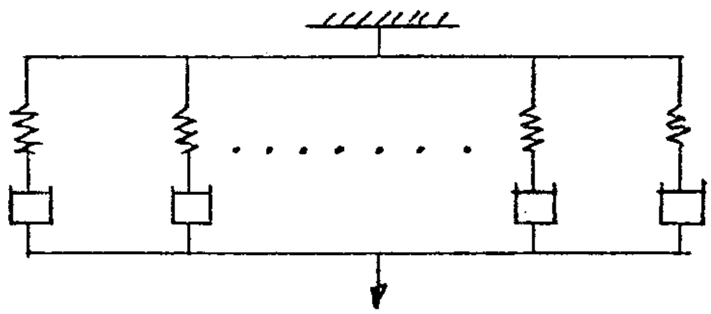
$$\frac{\varepsilon(t)}{\sigma_o} = J(t) = J_o + \int_0^\infty f_1(\tau)\left[1 - e^{-t/\tau}\right] d\tau + \frac{t}{\eta_o} \quad \text{. . . . . (15)}$$

$n \longrightarrow \infty$

where J_o corresponds to compliance of free spring and η_o to viscosity of free dashpot. J_o allows for discrete contribution with $\tau = 0$; sometimes termed the glassy compliance. May be inaccessible experimentally - however presence must be inferred otherwise instantaneous deformation would require infinite stress.

For relaxation,

generalized Maxwell model convenient:



$n \longrightarrow \infty$

$$\frac{\sigma(t)}{\varepsilon_o} = E(t) = \int_0^\infty f_2(\tau) e^{-t/\tau} \, d\tau + E_e \quad \ldots\ldots\ldots\ldots\ldots \quad (16)$$

allows for finite stress at long time.

Can also develop information for constant rate-of-strain and vibratory tests and develop, for example, from vibratory tests can develop either complex modulus or compliance.

By examining generalized expressions such as eq. (15) and (16), can be noted:

$$J(t) \longrightarrow J_o \quad \text{as} \quad t \longrightarrow 0$$

$$J(t) \longrightarrow \frac{t}{\eta} \quad \text{as} \quad t \longrightarrow \infty$$

$$\left|J^*\right| \longrightarrow J_o \text{ as } \omega \longrightarrow \infty$$

$$\left|J^*\right| \longrightarrow \frac{1}{\omega\eta} \text{ as } \omega \longrightarrow 0$$

Thus if $J(t)$ is plotted against time, and on the same graph $\left|J^*\right|$ is plotted against $\frac{1}{\omega}$, both curves will coincide for short times and again at long times; they may, however, differ in between.

$$E(t) \longrightarrow \frac{1}{J(t)} \text{ as } t \longrightarrow 0 \text{ or } t \longrightarrow \infty$$

Again at intermediate times $E(t) \neq$ exactly $\frac{1}{J(t)}$ for the generalized representation.

Effects of Temperature

Effects of temperature on viscoelastic behavior of materials inferred empirically originally. In 1953 P. E. Rouse* presented a theory which placed temperature dependence of viscoelastic properties on a more sound theoretical basis.

Essentially it is assumed that time and temperature are interchangeable for linear viscoelastic materials. (verified many times experimentally) As will be seen in the following discussion, a dimensionless factor

$$a_T = \frac{t_T}{t_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (17)$$

where: t_T is the time required to observe some phenomenon at temperature T, and t_o is the time required to observe the same phenomenon at a reference temperature T_o.

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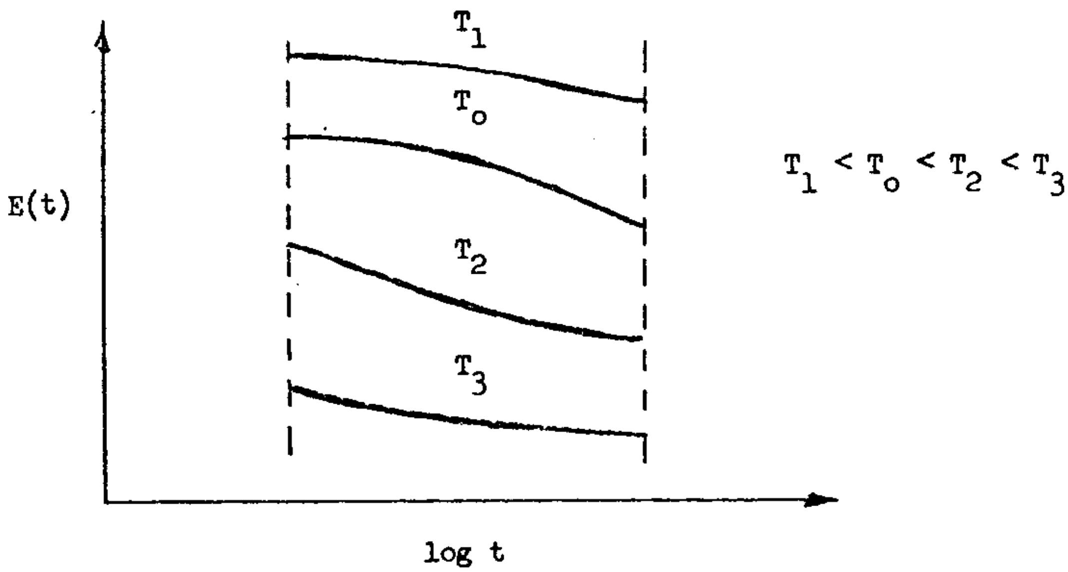
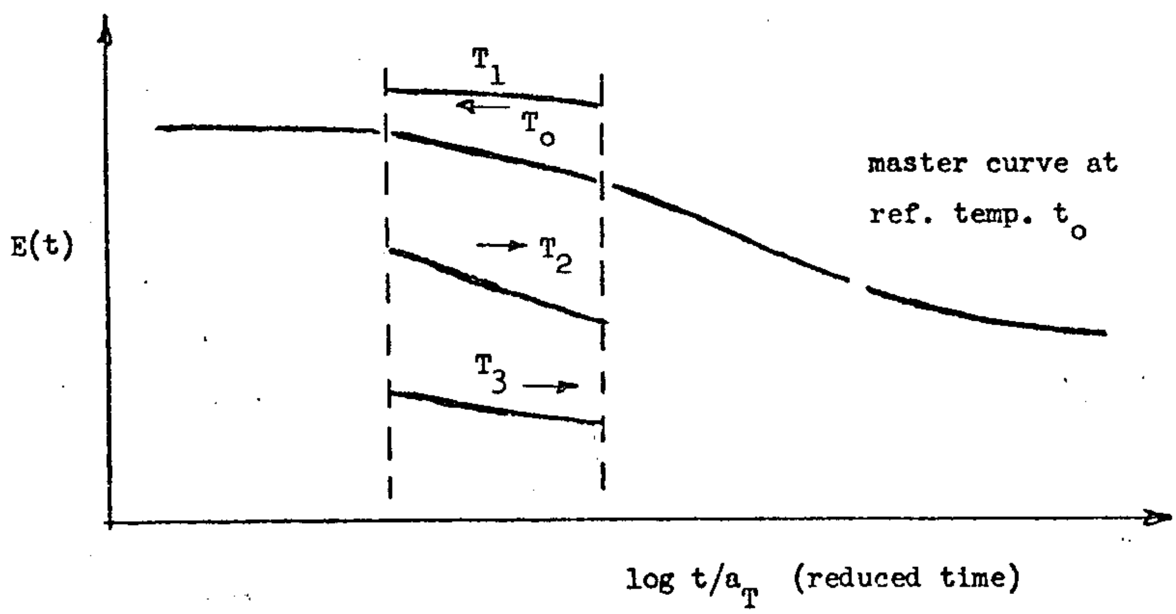
* See: Ferry, J. D., Viscoelastic Properties of Polymers, New York: John Wiley and Sons, 1961.

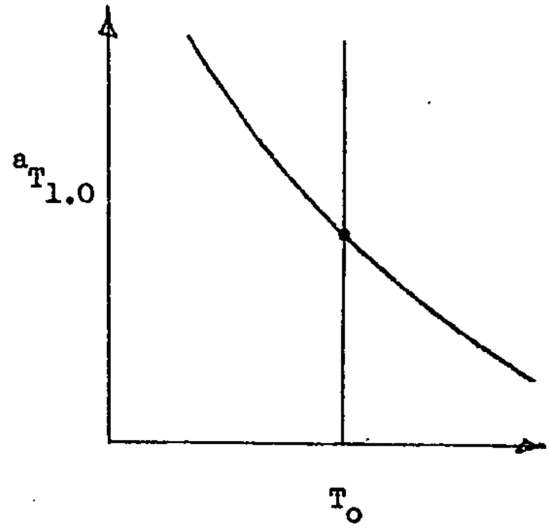
Consider the following: (assume that E(t) can be measured within a certain time range at different temperatures)



Through the use of the concept of interchangeability of time and temperature can shift all of the curves to the reference temperature (eg. T_o) and develop a master curve of E(t) covering many decades of time.



With the master curve at a particular temperature it is also necessary to have a plot of the shift factor a_T as a function of temperature. (that is the magnitude required to shift a curve at temperature T to reference temperature T_o so that the curves will match)



Strictly speaking in the previous case, E(t) in the reduced curve should also be multiplied by the factor $\frac{T_o \rho_o}{T \rho}$. Since, however, T is in ^oK, and ρ (density) changes little with temperature (at least for range of practical consideration in asphalt paving technology) this has been omitted.

For uncrosslinked polymers (e.g. asphalt might be considered in this category)

$$a_T = \frac{\eta \, \tau_o \, \rho_o}{\eta_o \, T \, \rho}$$

Thus if temperature dependence of viscosity is known, not necessary to obtain a_T empirically by shifting graphically as described above.

The type of approach described above has been used by Van der Poel (see ref. 1 f 18.) to develop a comparatively simple means for characterizing the effects of time and temperature on the behavior of asphalts over a wide time scale and large temperature range.

For this approach Van der Poel introduced concept of stiffness:

i.e $S = \frac{\sigma}{\epsilon}$ (at a particular time and temperature)

= tensile stress/total strain

To measure stiffness of asphalt Van der Poel used creep and vibratory tests.

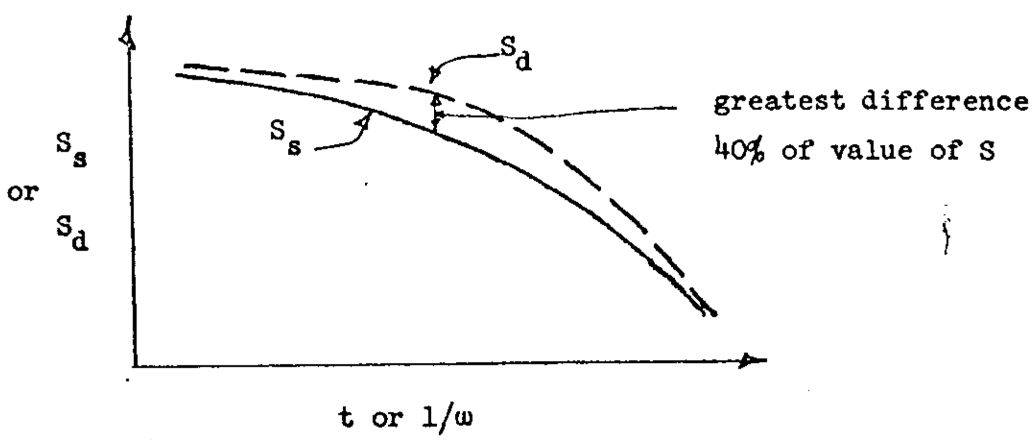
From creep tests, static stiffness obtained:

$$S_s = \frac{\sigma}{\epsilon} \quad (\epsilon \text{ a function of time})$$

From vibratory tests, dynamic stiffness obtained:

$$S_d = \frac{\hat{\sigma}}{\hat{\epsilon}} \quad \text{where:} \quad \left\{ \begin{array}{l} \sigma = \hat{\sigma} \text{ sim } \omega t \\ \epsilon = \hat{\epsilon} \sin(\omega t - \phi) \end{array} \right\}$$

For practical purposes Van der Poel noted that time and frequency are interchangeable and one could plot:



When material is elastic, S = E

" " " purely viscous, $\frac{d\epsilon}{dt} = \frac{\sigma}{\lambda}$ . . . . (20)

and $\lambda = 3\eta$

For static experiment (creep)

$$\epsilon = \frac{\sigma}{3\eta} \cdot t$$

$$S = \frac{3\eta}{t}$$ (i.e. stiffness proportional to time of loading for viscous material)

For dynamic experiment (vibratory loading)

$$\lambda \hat{\varepsilon} \omega \cos (\omega t - \phi) = \hat{\sigma} \sin \omega t$$

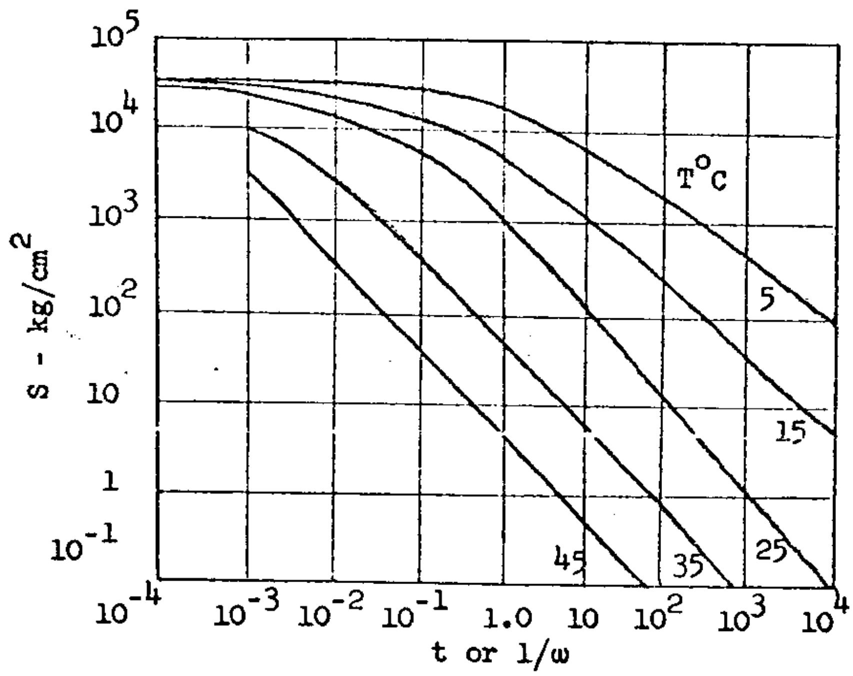
(obtained by substituting eq. 18 and 19 in 20.)

This condition is satisfied if:

$$\lambda \hat{\varepsilon} \omega = \hat{\sigma} \text{ and } \phi = \frac{\pi}{2}$$

$$\therefore S = 3\eta\omega \text{ ; which corresponds to } S = \frac{3\eta}{t}$$

when $\frac{1}{\omega}$ substituted for t.



Asphalt:

R and B temp 66°C

P.I. = - 2.3

Dependence of stiffness on time of loading at various temperatures for asphalt if low P.I. (after Van der Poel)

Referring to Van der Poel's data (figure on previous page), curves can be shifted to obtain a "master" curve.

$$\therefore \quad S = f \left\{ - \log \frac{t}{t_o} + \chi (T) \right\}$$

where t_o = some constant with time dimension and f and χ are empirical functions.

Van der Poel also noted that one could plot stiffness vs $T_{R \text{ and } B} - T$ (where $T_{R \text{ and } B}$ = ring and ball softening pt. temp.)

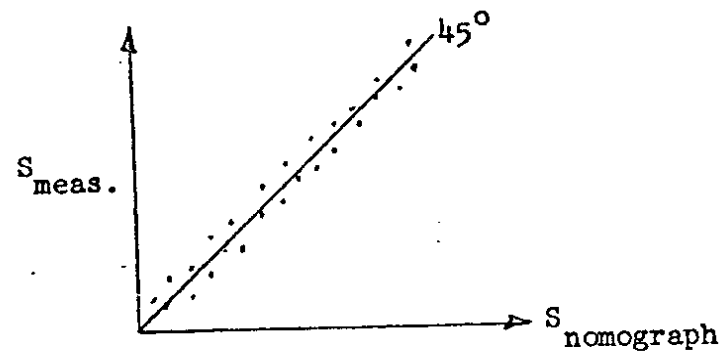
As with log penetration vs T_R and B^{-T}, curves for asphalt with same P.I. coincide.

$$\therefore \quad S = f\left\{ -\log \frac{t}{t_o} + C \cdot \phi\,(T_R \text{ and } B^{-T}) \right\}$$

and t_o, C, ϕ, and f depend solely on rheological character of asphalt.

He also found that it was possible to use same ϕ by choosing C as a function of rheologic type of asphalt (in this case the P.I.). The Nomograph for stiffness resulted from these considerations.

Validity of stiffness nomograph checked, e.g. by use of rotating cylinder viscometer.



deviations seldom exceeded a factor of 2.

This difference of 2 in stiffness, Van der Poel notes, corresponds to temperature difference of 2°. In general, he noted fairly good correlation from 100°C above to 200°C below R and B temperature.

On nomograph note that:

(1) at low temperatures $\quad S \longrightarrow 3 \times 10^9\ N/m^2$

(2) viscosity point indicated at t = 3 seconds

$$\left(S = \frac{3\eta}{t}\ ; \quad \text{when } t = 3,\ |S| = |\eta| \right)$$

(3) also note that $1\ N\ sec/m^2 = 10$ poises

Van der Poel concluded from his study that:

1. <u>Stiffness</u> modulus depends on:

   (a) time of loading or frequency

   (b) temperature

(c) hardness of asphalt

(d) rheologic type of asphalt

2. Hardness of asphalt can be completely characterized by ring and ball temperature and rheologic type by penetration index (P.I.).

3. At low temperatures all asphalts behave elastically with $S = 3 \times 10^{10}$ dynes/cm^2.

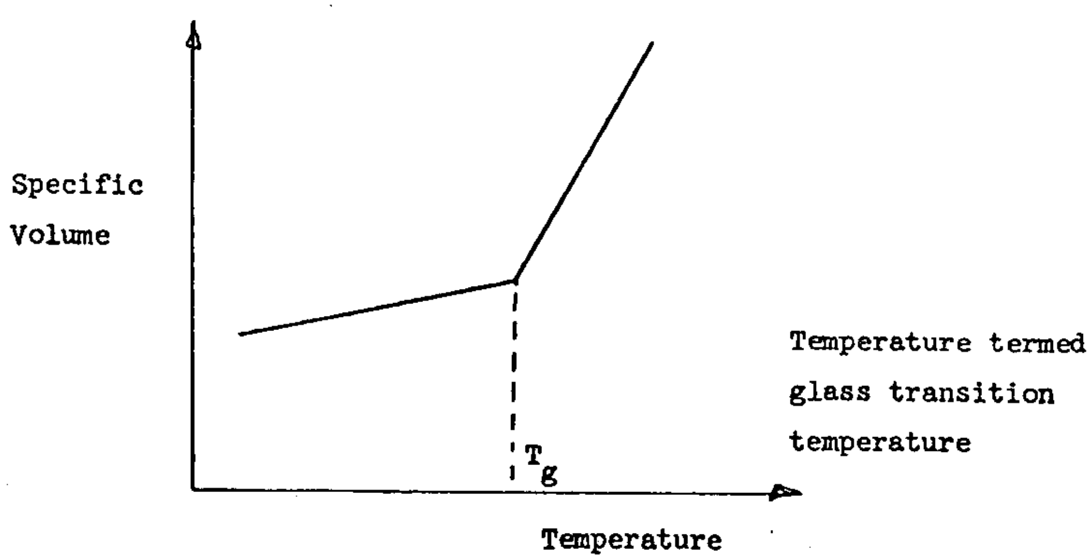
(while not discussed herein)

4. Penetration corresponds to stiffness @ 0.4 sec.

5. Frass test is essentially an equistiffness test and gives temperature at which $S = 1.1 \times 10^9$ dynes/cm^2 @ 11 sec.

TRANSITION FROM VISCOELASTIC TO GLASSY (ELASTIC) STATE

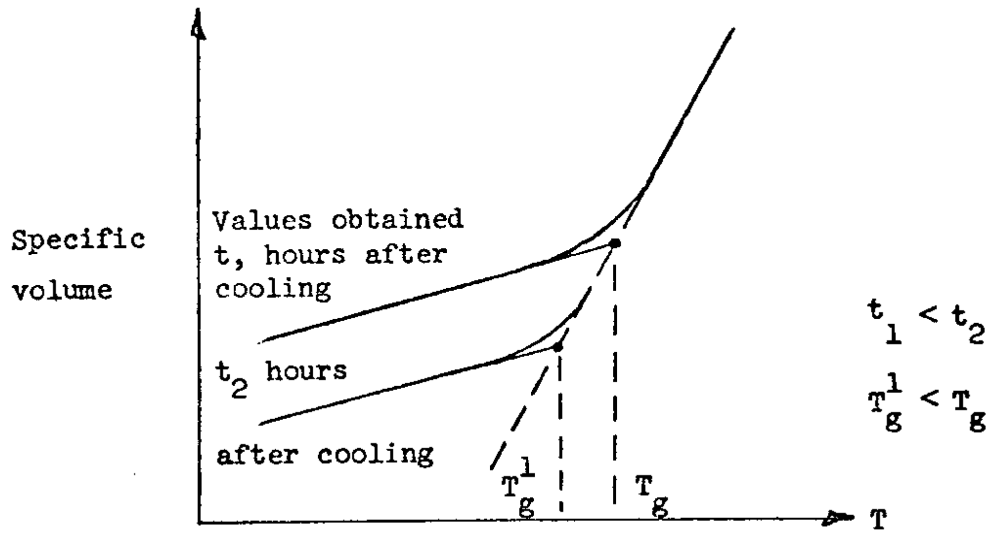
By cooling material (polymer) and measuring specific volume, following type of relationship obtained:



In the glassy region (for amorphous polymers) time effects are not very pronounced so that we can speak of a quasi-static modulus, E_g.

Should be noted that T_g as indicated on previous page depends on time scale of volumetric measurement. Generally slower measurement leads to a lower value of T_g.

i.e.



Glass transition temperature useful because A_T can be determined if T_g is known. Then, as noted earlier for an uncrosslinked (amorphous) polymer,

$$A_T = \frac{\eta T_o \rho_o}{\eta_o T \rho}$$

Thus, viscosity can be determined at any temperature. Williams - Landel - and Ferry (WLF) have suggested the following equation for A_T as being applicable to amorphous polymers above their glass transition:

$$\log A_T = \frac{-\ 8.86\ (T - T_s)}{101.6 + T - T_s}$$

where: T_s - reference temperature, empirically chosen

and $T_s \doteq T_g + 50^\circ C$

Actually, constants 8.86 and 101.6 in the above equation depend somewhat on nature of polymer.

Wada and Hirose* have presented data of T_g determinations for various asphalts. Illustrate that T_g a function of asphaltene content (next page). They also suggest that WLF equation applies if T_s is $56^{\circ}C$ rather than $50^{\circ}C$ above T_g.

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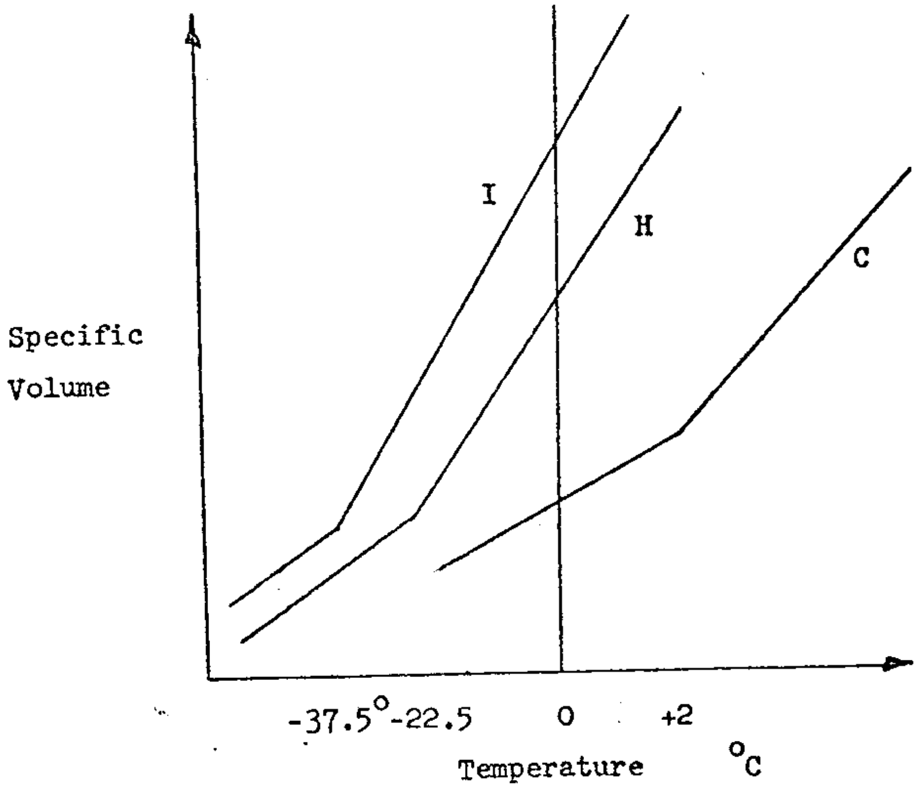
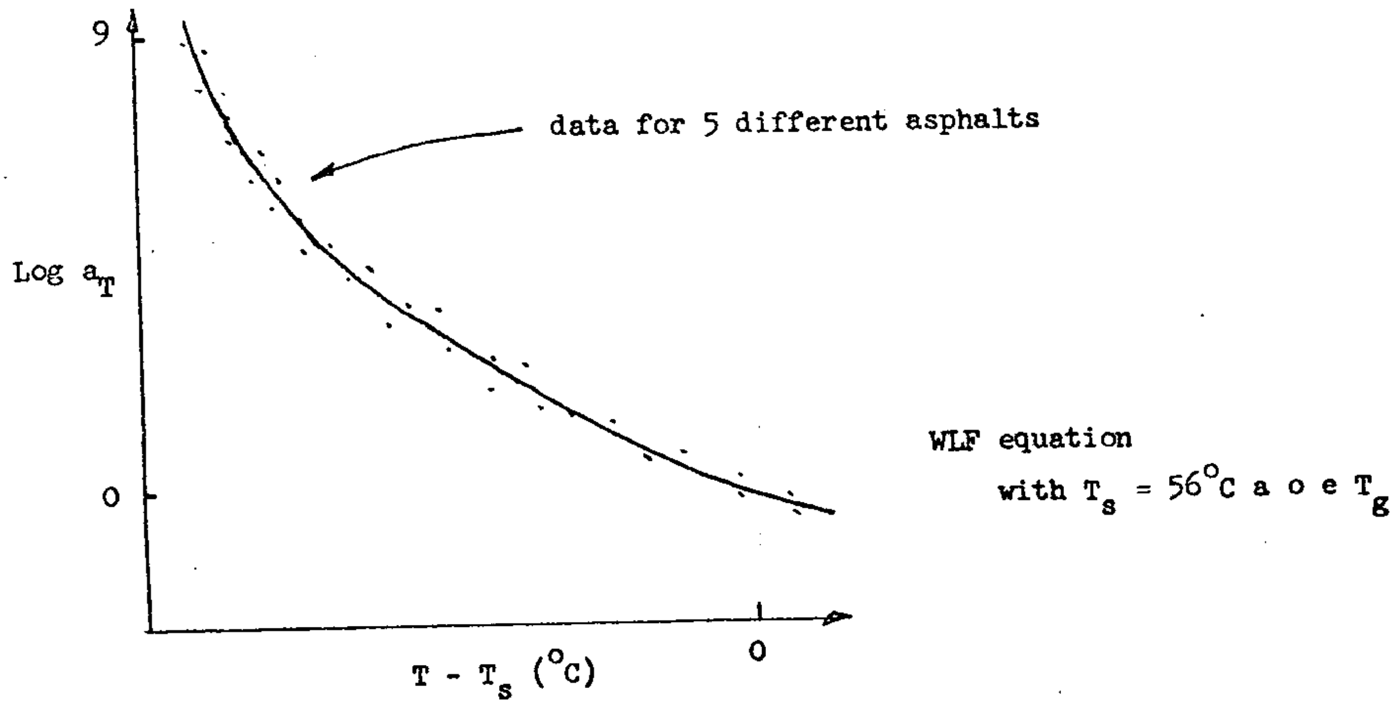
*Wada, Y. and H. Hirose, "Glass Transition Phenomena and Rheological Properties of Petroleum Asphalt", Journal, Physical Society of Japan, Vol. 15, No. 10, Oct. 1960, pp 1885-94.



Sample	Asphaltene Content
C	61.9%
H	28.6
I	0
	(by wt)



WADA AND HIROSE INFORMATION

MATERIALS BEHAVIOR AS RELATED TO DESIGN LIFE

Pavement design, requires that engineers have the ability to analyze their pavement structure in terms of significant system parameters. It is necessary that such analyses incorporate essential features of observed pavement performance and appropriately measured values of the parameters to make the necessary quantitative evaluations required for design. It is generally recognized that these parameters, with their interrelationships, are complex. Several attempts have been made to formulate, in a systematic manner, pavement design systems which bring these factors together as a part of the development of improved methods of pavement design; methods which eventually may have the capability to

i) accommodate the everchanging requirements of loading

ii) better utilize available materials

iii) accommodate new materials which might be developed

iv) better define the role of construction.

Figure 1 illustrates the complexity of such an approach.

This paper outlines a few subsystems to examine specific distress modes. The subsystems are:

i) fatigue in the asphalt bound layer due to repetitive traffic loading

ii) permanent deformation in the pavement structure

iii) fracture due to braking and acceleration forces or due to thermal stresses

DISTRESS MODES

As in Figure 1, the limiting response of the pavement system to various inputs has been termed distress. For convenience, the various distress mechanisms have been grouped into three categories which either by themselves or in combination can lead to a reduction in pavement serviceability with time. Essentially, the goals of both pavement and asphalt mixture design are to minimize the various distress manifestions shown in this figure to permit the pavement structure to serve its intended function for some specific time period or to permit it to carry some estimated number of load applications. The three distress modes are fracture, distortion, and disintegration. Figure 2 lists these modes and their various distress manifestations as well as the distress mechanisms.

FATIGUE

Fatigue is the phenomena of repetitive load-induced cracking due to repeated stress or strain level below the ultimate strength of the material.

One form that the fatigue subsystem for considering the fatigue mode of distress can take is as shown in Figure 3.[1]

TRAFFIC CHARACTERISTICS

An estimate of the traffic and wheel load distribution to be served by the proposed facility is required.

For a Highway pavement, the following traffic information should be determined:

i) the number of vehicles in each load and axle classification

ii) wheel and axle configurations (dual or single tires and single or tandem axles)

iii) the distribution of truck traffic throughout the day, month and year

iv) contact (or tire) pressures of the various classes of vehicles

v) vehicular velocities; and lane distribution of truck traffic for multilane facilities

For airfield pavements, the required information includes:

i) gear configurations of representative aircrafts using the facility

ii) contact (or tire) pressures of the various aircraft

iii) aircraft weights as affected by length of flight and takeoff and landing operations

iv) daily and seasonal variations in aircraft movements

v) lateral distribution of loads on taxiways and runways and longitudinal load distribution on runways

vi) aircraft velocities

Methodology for estimating traffic is discussed by Hudson[2] and Deacon[3] and they suggested a procedure for discretizing the spectrum of loads to make the traffic factor more manageable in the analysis stage. To simplify the process even further, Havens, Deen, and Southgate[4] use the concept of equivalent axle loads, reducing all traffic to a common parameter--passages of an 18,000-16 axle load. Witczak[5] had made use of the same concept for airfield pavements defining all aircraft in terms of equivalent passes of a fully loaded DC-8-63F aircraft.

ENVIRONMENTAL CONDITIONS

Temperature

i) In the case of an asphalt concrete pavement, since the response of asphalt concrete to load is dependent on temperature, distributions of temperature within the asphalt bound layer must be obtained. Such distributions can be determined using a form of the heat conduction equation[6]. It is possible to include not only daily temperature variations but also the effects of solar radiation, sky or cloud cover and wind velocity in this determination.

Moisture

ii) In addition to temperature effects, other environmental influences must also be taken into account. Among the most important of these is that of water, particularly its influence on the response of paving materials to stress. Not long ago, a method was proposed for fine grained soils whereby the influence of the environment (particularly as it influences water content) might be accounted for utilizing considerations of soil moisture suction[7]. Data obtained by Dehlen[8] for specimens obtained from the San Diego Test Road indicated a distinct relationship between suction and resilient modulus. It should be noted that the concept of suction permits an assessment of distortion of the pavement structure due to volume change in the subgrade.

MATERIAL CHARACTERIZATION

Fatigue tests can be conducted by several methods and various specimen size. The most common test method is a repeated load flexure device with beam specimens. Split tensile test (or Repeated load Indirect Tensile tests) have also been used. (See Yoder and Witczak Chapter 8, Pages 267-275; 282-289)

Fatigue testing may be conducted under two types of controlled loading--controlled stress or controlled strain. In the controlled-stress (load) mode, a constant load is continuously applied to the specimen and because of the progressive damage to the specimen, a decrease in stiffness results. This in turn causes an increase of the actual flexural strain with load applications. In the controlled strain (deflection) approach, the load is continuously changed to yield a constant beam deflection. This results in a stress that continuously decreases with load applications.

In the design subsystem of Figure 3, some measure of the fatigue response of asphalt mixtures is required. For thick asphalt pavement layers (more than 6 inches), this response is measured by means of Controlled-stress mode of loading. For comparatively thin asphalt bound layers (less than 2 inches), the controlled-strain mode is more appropriate. At intermediate thicknesses, the probable fatigue response is governed by something intermediate to these two test modes. Differentiation between the two modes of loading can be placed on a more quantitative basis by introducing a parameter termed the Mode Factor[9] and defined as:

$$MF = \frac{|A| - |B|}{|A| + |B|}$$

where:

MF = mode factor

A = percentage change in stress due to a stiffness decrease of C percent

B = percentage change in strain due to a stiffness decrease of C percent

C = an arbitrary but fixed percent reduction in stiffness

In the controlled-stress mode of loading, the mode factor would have a value of -1, whereas for the controlled-strain mode of load the mode factor would assume a value of +1.

In laboratory fatigue testing, it has been demonstrated that the influence of stiffness of the asphalt concrete is an extremely important consideration. Results of fatigue tests may be plotted either in the form of stress or strain versus fatigue life (Figure 4). This results in a relationship represented by the equation:

$$N_f = K \left[\frac{1}{\varepsilon mix} \right]^n$$

where:

N_f = stress applications to failure

ε_{mix} = tensile strain repeatedly applied to the mix

$k_1 n$ = constants depending on mixture characteristics

The coefficient n in the above equation appears to be dependent on mixture stiffness. For the range of stiffness encountered in practice, n varies in the range 2 to 6.

Fatigue Life Estimation

In practice, pavements are subjected to a range of loadings; accordingly, a cummulative damage hypothesis is required since fatigue data defined by the above equation are usually determined from the results of simple loading tests. One of the simplest of such hypothesis is the linear summation of cycle ratios, which in simple form may be stated as

$$\sum_{i=1}^{i} \frac{n_i}{N_i} = D$$

where n_i = number of applications at strain level i

N_i = number of applications to cause failure in simple loading at strain level i, and

D = total cummulative damage.

In this relationship, failure occurs when D equals or exceeds 1.0. Thus the design procedure becomes one of checking the particular section to ensure that D is equal to or less than unity for the anticipated design conditions. When the value of D is considerably less than one, the section may be underdesigned; when D is greater than one, a redesign or reanalysis may be in order. As Deacon notes, various temperatures as well as loading conditions can be considered and thus make the procedure adaptable to any environmental condition.

Terrel [10] presents a number of examples illustrating the applicability of such an approach to analyze fatigue distress occurring in either trial or in-service pavements. The comparisons presented lend support to the use of this procedure for design purposes now.

Havens, Deen, and Southgate (4) in their procedure, described the fatigue life and thus the design section as associated with the number of repetitions of 18 kips single axle load; and some equivalency between the other loads and the 18 kips value is thus established. Witczak[5] also made use of the linear summation of cycle ratios to convert traffic to equivalent passes of a DC-8-63F aircraft for design purposes.

It must be emphasized that the estimate made by this technique is associated with no specific amount of cracking. If the analysis is based on laboratory fatigue tests, the traffic will be that associated with crack initiation and will provide a slightly conservative estimate.

PERMANENT DEFORMATION

Permanent deformation can be treated as follows:

i) traffic induced permanent deformation

ii) non-traffic induced permanent deformation

Permanent deformation due to moving traffic can be defined or at least limited to time-dependent distortion or volume change or both caused by densification of one or more layers within the pavement system. Deformation can take place in one or all layers although it is noticeable only at the surface in the form of ruts, lateral and longitudinal corrugations, shoving, and other movements.

Other non-traffic induced permanent deformation are those having their source or cause of deformation from hydrothermal volume changes in elements of the pavement structural section and the foundation thereof. Other sources may be problems associated with distortion due to differential settlement within embankments or displacement (creep) within embankment foundations.

Figure 1 shows that distortion or permanent deformation is one of the limiting distress response outputs of the pavement sytem. Within this framework, those items that can be considered under permanent deformation are as follows:

1. Excessive loading
2. Time-dependent deformation (creep)
3. Densification
4. Consolidation, and
5. Swelling

Of these, item 2 appeared to be of most concern, whereas items 3,4, and 5 can be lumped together as volume change. Item 2 and 3 could be taken as the major causes of traffic-induced permanent deformation while items 3,4 and 5 can be the major causes of the non-traffic induced permanent deformation.

TRAFFIC INDUCED PERMANENT DEFORMATION[11]

Material Characterization

Research and experience have shown the response of most pavement materials to be time-dependent and to be probably affected by the properties of materials themselves, their relationships or proximity to other materials in the system and the usual factors of load, environment, and so forth. Conventional materials and their suggested form of characterization follow:

1. Asphalt mixtures--linear viscoelastic
2. Granular bases or subgrades--assumed to be elastic or linear viscoelastic
3. Cohesive subgrade--linear viscoelastic
4. Other materials including portland cement concrete and cement-treated base--assumed to be elastic.

Characterization of these materials in the laboratory for input parameters can best be accomplished by using a triaxial test apparatus. The types of tests deemed suitable would include at least the following:

1. Constant stress or strain (creep)

2. Sinusoidal, and
3. Repeated load

It is recognized that one or more of these tests may be used to determine both the time-dependent and the volume change responses to loading and environment.

STRUCTURAL ANALYSIS AND PREDICTIVE TECHNIQUES

A method of computing or determining pavement behavior in terms of stress, strain, deflection, or permanent deformation is essential to the design or analysis process. The following is a brief summary of the status of present methodology.

EXISTING TECHNIQUES

The currently available methods based on stability criteria tend to preclude permanent deformation at least for conventional materials and designs. However, it is suggested that in the California method, if the resistance value at the subgrade, R, is 10 and the asphalt concrete layer on top varies from 5 to 7 inches, no traffic-induced permanent deformation is to be anticipated. In order to prevent traffic-induced permanent deformation, the subgrade needed to be strengthened. To minimize the strain in the subgrade requires that the subgrade have a high bearing value (e.g. R=10) or be stabilized.

QUASI-ELASTIC METHOD

The method developed by Shell suggests that, if the strain at the top of the subgrade does not exceed 6.5×10^{-4}, no permanent deformation could be anticipated for 10^6 repetitions of an 18-kip axle load. This approach is based upon the use of elastic theory and the results of plastic strains determined by repeated load laboratory tests on pavement materials. The

AASHO Interim Guide and the Kentucky method are also based on similar principles.

LINEAR VISCOELASTICITY FOR LAYERED SYSTEMS

In order that we can estimate the manner in which deformation accumulates in flexible pavements, a model is needed to account for the manner in which this deformation accumulates as a function of load, environment, and material variables. Specifically, the model should be able to account for the following variables:

1. Time-dependent behavior of materials
2. Temperature-dependent behavior of materials
3. Magnitude, duration, and number of repetitions of the loads
4. Influence of moisture changes

A linear viscoelastic model of layered systems that can account for variables 1,2 and 3 have been developed. This operational model requires that the creep properties of materials be given in the form of creep compliance functions. It provides the total deflection and the permanent recoverable deformation. The influence of temperature and its variation can be accounted for only if the time--temperature superposition principle is assumed to be valid. The model accounts for randomness of load, temperature, and material properties in a simulative manner by using random number generators.

With regard to all three approaches, it can be recognized that the first and second are primarily methods of preventing excessive deformation in the form of rutting. However, the quasi-elastic approach has also been used in approximating the amount of rutting to be expected. The third method, based on linear viscoelastic theory, is an attempt to actually permit prediction of accumulated deformation.

FRACTURE

Pavements are designed to resist fracture from a single load application as well as from many load applications. Traffic loading conditions leading to fracture include those associated with accelerating and decelerating traffic and occasional overload axles.

In addition to loading, pavements are subjected to environmental influences. Such factors, acting alone or in combination with load, can lead to distress. Potential contributing factors to non-traffic load associated cracking of asphalt pavements may be summarized as:

1) Volume changes in the mix itself due to temperature changes, absorptive aggregates, and volatilization of asphalt,
2) Volume changes in underlying materials due to moisture changes, temperature changes, and curing of cement.

FRACTURE CHARACTERISTICS

As with stiffness, the fracture characteristics of asphalt paving mixtures are dependent on temperature and rate of loading. As temperature decreases or rate of loading increases, the fracture strength increases. Heukelom [12] has developed a procedure to estimate the fracture strength of a mix from a knowledge of the properties of the asphalt contained therein. He has shown that

$$\left(\sigma_b\right)_{asph} = \frac{1}{M_t} \left(\sigma_b\right)_{mix}$$

where

$\left(\sigma_b\right)_{asph}$ = fracture strength of asphalt at a particular time of loading and temperature

$\left(\sigma_b\right)_{mix}$ = fracture strength of mix under corresponding conditions

M_t = mix factor (function of filler content, aggregate type, and asphalt content)

This procedure minimizes the amount of laboratory tests to be performed.

According to Heukelom, the fracture strength of asphalt corresponding to a particular time-of-loading and temperature can be estimated from its penetration and ring and ball softening point. Hence for a particular mix, from one test condition the mix factor M_t can be deduced. An estimate of the fracture strength under other load conditions can then be determined utilizing his procedure for estimating the fracture strength of the asphalt.

FRACTURE UNDER LOAD:

As with other performance aspects, development of stresses in the asphalt-bound layer of such magnitude that fracture might result is dependent on the characteristics of the other components of the pavement structure as well as the asphalt mixture itself. Use can be made of layered-system theory to obtain an estimate of load stresses resulting from vertical loading conditions for comparison with fracture strength.

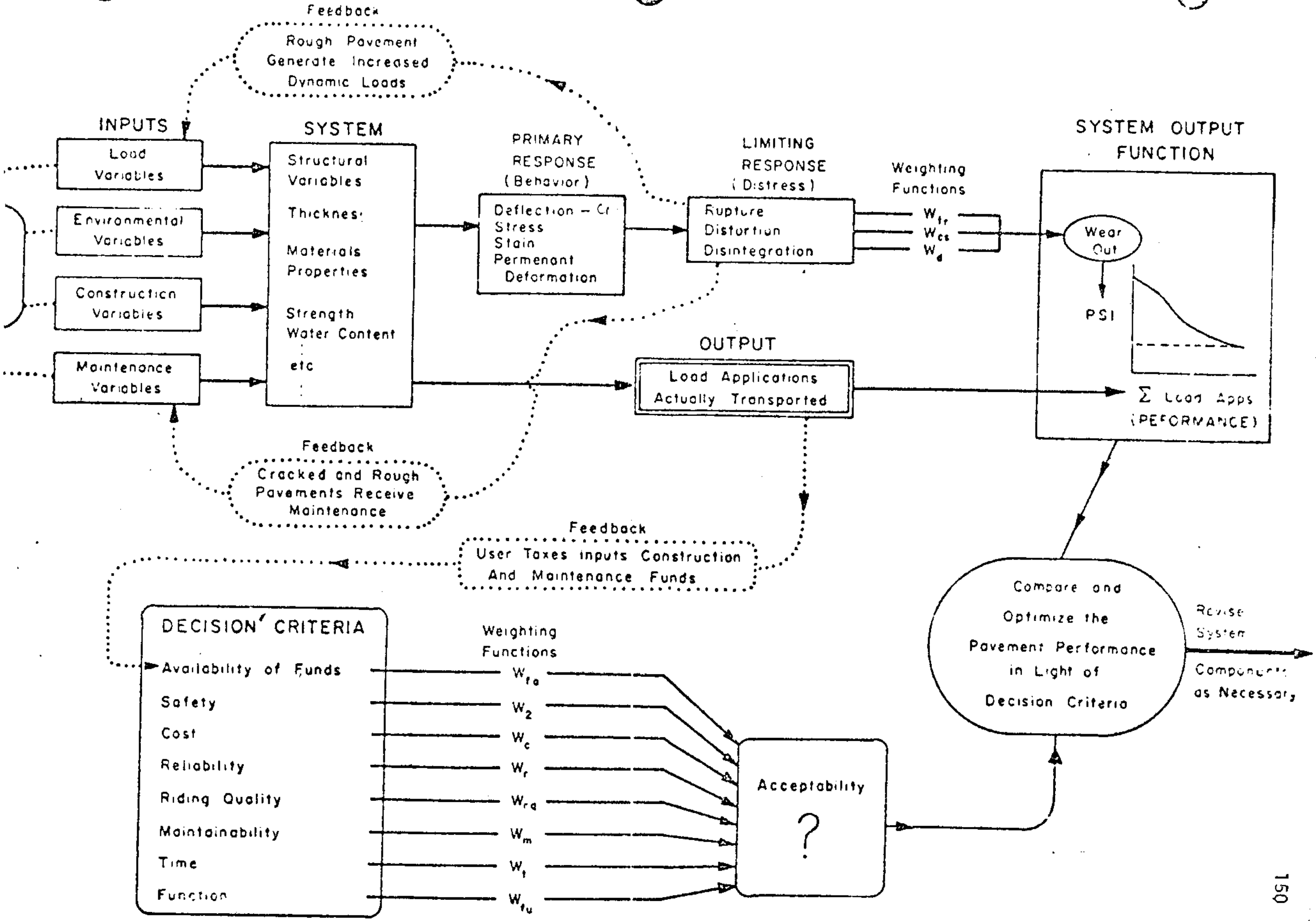
A potentially more common condition leading to fracture is that resulting from sheer stresses applied at the pavement surface (e.g., braking forces). McLeod[13] in his analysis indicated that as the asphalt layer thickness is increased, the resistance to fracture is increased. Although somewhat limited, Verstraeten[14] also emphasized that a combination of vertical and shearing loads applied at the surface of the pavement may develop stresses within the asphalt bound layer or at the interface between layers which may exceed the strength of the material or the bond between layers.

A recent study at Ohio State University[15] utilized the principle of fracture mechanics to explain the mechanism of damage and the prediction of the fatigue life of pavements. This method considers fatigue of asphaltic mixtures as a process of damage where under a given state of stresses damage grows according to a crack propagation law from an initial stage to a critical and final level.

ENVIRONMENTAL INFLUENCES

Fracture may result from environmental influences as well as load stresses. Of the environmental factors, only that of temperature is amenable to analysis at present. Monismith[16] have calculated stresses in an asphalt concrete layer due to temperature changes at the surface of the pavement. He showed that only under the very severe temperature conditions were stresses developed which would exceed the expected values for fracture strength. It should be emphasized that these stresses occur at the surface of the pavement. Because of the attenuation of temperature change with depth, the tensile stresses due to temperature will decrease within the pavement. This point is important when considering combination of load and temperature stresses, since the tensile stresses due to load are at a maximum on the underside of the asphalt layer. It is possible, however, that a combination of tensile stress due to load with that due to temperature may lead to fracture under certain circumstances.

Temperature stresses are influenced by the characteristics of the mixture. For example, a mixture in which essentially elastic behavior is induced at a comparatively high temperature would be more prone to fracture at low temperatures than one which develops its elastic behavior at comparatively lower temperatures. This behavior in turn may be influenced by the characteristics of the asphalt, with a softer, less temperature-susceptible material appearing to contribute to better performance at these lower temperatures through its influence on the low-temperature theologic behavior of mixtures.



PAVEMENT MANAGEMENT SYSTEM

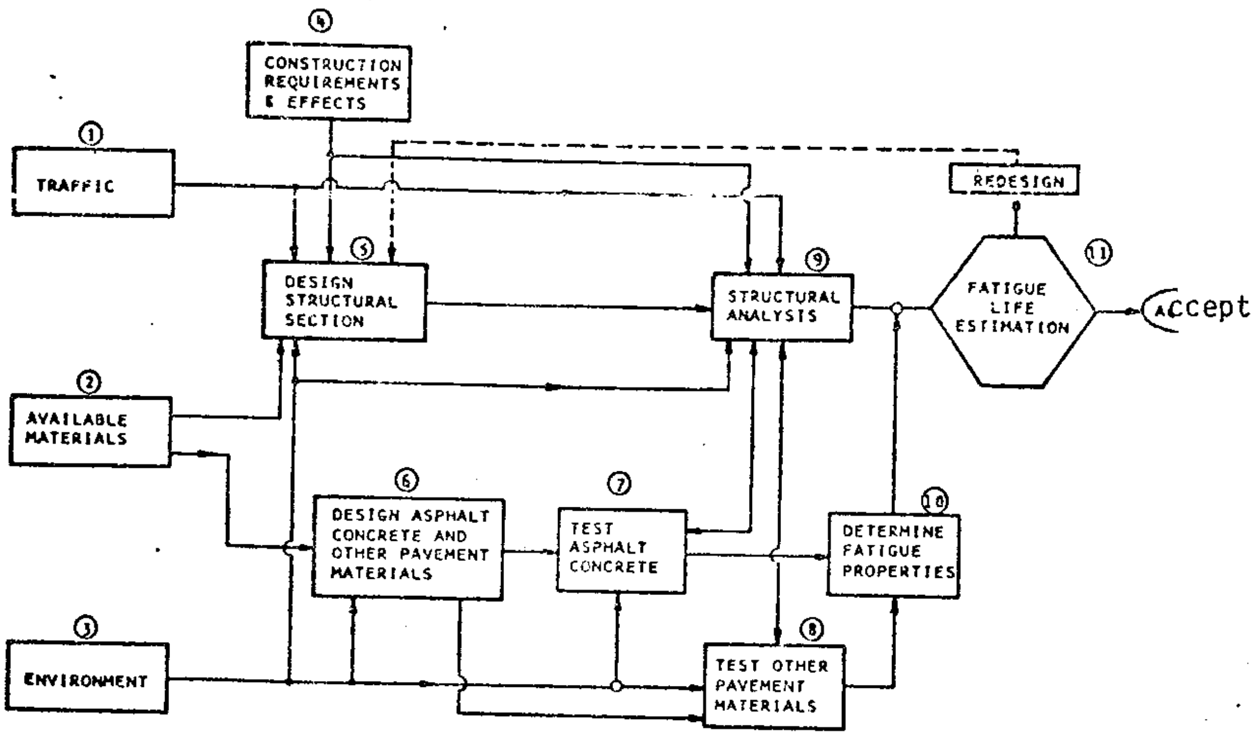
FIGURE 1 - Hudson, Finn McCullough Nair

Distress Mode	Distress Manifestation	Examples of Distress Mechanism[1]
Fracture	Cracking	Excessive loading Repeated loading (i.e., fatigue) Thermal changes Moisture changes Slippage (horizontal forces) Shrinkage
	Spalling	Excessive loading Repeated loading (i.e., fatigue) Thermal changes Moisture changes
Distortion	Permanent deformation	Excessive loading Time-dependent deformation (e.g., creep) Densification (i.e., compaction) Consolidation Swelling
	Faulting	Excessive loading Densification (i.e., compaction) Consolidation Swelling
Disintegration	Stripping	Adhesion (i.e., loss of bond) Chemical reactivity Abrasion by traffic
	Raveling and scaling	Adhesion (i.e., loss of bond) Chemical reactivity Abrasion by traffic Degradation of aggregate Durability of binder

(1) Not intended to be a complete listing of all possible distress mechanisms.

Fig. 2 - Categories of Pavement Distress. (After Hudson, Vallerga, Nair, McCullough)

Figure 3. Diagram of a fatigue subsystem.



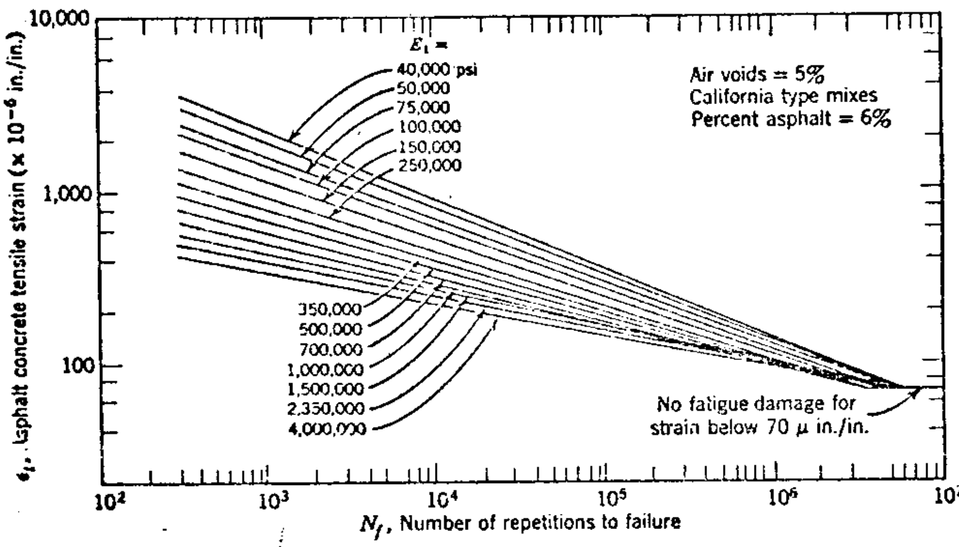


Figure 4.. Typical fatigue criteria. (From Monismith.)

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FLEXIBLE PAVEMENT DESIGN AND MANAGEMENT SYSTEMS

Systems engineering is a broad concept with many definitions. Basically, it is a codified procedure for attacking complex problems in a coordinated fashion to permit realistic design developments that can be justified in the face of certain decision criteria. Highway pavements can be viewed as complex structural systems involving many variables, e.g., combinations of load, environment, performance, pavement structure, construction, maintenance, materials and economics. In order to design, build and maintain better pavements, it is important that most aspects of the pavement system be completely understood and that design and research be conducted within a systems framework.

Too often in the past the narrow view of the pavement design problem has produced unsatisfactory methods of constructing, designing, maintaining, and evaluating pavements. The narrow concepts of design previously used will not suffice for high-speed, high volume, modern pavement facilities. Figure 1 shows the steps involved in systems engineering. A concise statement of the objectives of the system is a necessary step toward a solution. With these objectives in mind, it is possible to establish systems requirements or define the problem. From these requirements, a well-defined model of the problem can be developed and alternate solutions can be generated. From the alternate solutions, the engineer can select the best solution, based on some type of decision criteria, and this solution can be implemented by construction. The method does not end here, because it is necessary to obtain feedback information and check the performance data that have been obtained from the constructed pavement. These feedback data make it possible to modify designs and ultimately to modify the method, if necessary. There is no one, unique, overall solution, but, in general, this approach can be helpful in solving the problem.

There are many ways of modeling the problem, including physical, conceptual, and mathematical models. The conceptual model of the pavement system is shown in Fig. 2.

It includes not only a particular set of mathematical models or graphs, but also decision criteria and a systems output function. In addition, an implementation plan, equipment, and personnel necessary to implement the system are essential parts of the over-all systems model.

SAMP

Although conceptualizing the over-all pavement system was essential to solving the problem, it was necessary from an application standpoint to develop an operational system. After a review of the efforts of researchers in the area of applying systems concepts to pavement design, it seemed desirable to modify and extend the efforts of Scrivner, McFarland and Carey at the Texas Transportation Institute, who had developed the first known computer-oriented operational system for design of flexible pavement. Thus, Systems Analysis Model for Pavements (SAMP) was developed as an extension of the algorithm conceived and developed at the Texas Transportation Institute. SAMP is an operational systems model; i.e., a set of models with its pertinent computer program for making solutions. For easy identification, because improvements were constantly being added to the system, numbers were added to the acronym (i.e., SAMP1, SAMP2, ...SAMPn) to designate subsequent versions of the same basic program to which improvements had been made.

The purpose of the SAMP systems analysis is to design from available input data a pavement that can be maintained above the specified minimum serviceability over the specified design period at a minimum over-all cost. The computer program provides the decision maker with a set of feasible pavement designs arranged in some priority order. Other pertinent information necessary for use in making rational design decisions is also provided.

SAMP5

Seven classes of input variables are required by the program. Each class of variables is important in the solution of a problem by the computer. The classes are as follows:

1. Material Properties
2. Program Control and Miscellaneous Variables
3. Environmental and Serviceability Parameters
4. Load and Traffic Variables
5. Constraint Variables
6. Traffic Delay Variables
7. Maintenance Variables

Figure 3 shows a typical computer listing of input data for the SAMP5 program and Figure 4 is a typical computer output of SAMP5. Figure 5 is a block diagram of SAMP pavement system.

Although the SAMP5 program used up to 100 input variables that were thought to cover the range of variables normally considered in the pavement design process, it still needed to be implemented; that is, to be applied to actual pavement design problems. If discrepancies were noted between the program and practice then the program needed to be changed to reflect as closely as possible the real decision-making process. Full implementation of the computer program required detailed descriptions of how it was to be used, how data was to be input, and how data was to be obtained from the field using data feedback storage systems.

SAMP6

The primary objectives of this program were the further development of the SAMP5 program to the field application stage and its pilot testing in one or more state highway departments. It was anticipated that meeting these objectives would involve:

1. Pilot testing SAMP5, including a sensitivity analysis on one or more state highway departments using the current pavement structural design procedure of the test state as the structural subsystem.
2. Revising the working system as necessary in accordance with the experience gained during pilot testing.
3. Finalizing the SAMP5 working system as a pavement design and management tool,

including the preparation of detailed descriptions for the User's Guide, input forms, and feedback storage systems.

4. Determining research needs in each of the subsystems of SAMP5, using sensitivity analysis as needed.

The SAMP6 program requires twelve classes of input variables:

1. Program Control and Miscellaneous Input
2. Environmental and Serviceability Variables
3. Traffic and Reliability Variables
4. Constraint Variables
5. Traffic Delay Variables
6. Maintenance Variables
7. Cross-section, Cost Model, and Shoulder Variables
8. Tack Coat, Prime Coat, and Bituminous Materials Variable
9. Wearing Surface Variables
10. Overlay Variables
11. Pavement Material Variables
12. Shoulder Layer Material Variables.

The SAMP6 computer program contains the MAIN program, nine subroutine programs, and four function subprograms. Table 1 gives a cross-reference listing of the SAMP6 MAIN program and subprograms.

Figure 6 is an example summary of the input data and Figure 7 is an example output summary of an optimum design strategy for a four-layer system.

Figure 8 is an example output summary of the better over-all designs.

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8

TABLE 1

CROSS-REFERENCE LISTING OF SAMP6 MAIN PROGRAM AND SUBPROGRAMS

CALLED PROGRAM	MAIN	CALC	DESTYP	HEADNG	INCOST	INPUT	OUTPUT	OVRLAY	PUPY	RMAINT	SOLVE2	SUMARY	TIME	USER
CALC													X	
DESTYP	X													
HEADNG			X			X						X		
INCOST											X			
INPUT	X													
OUTPUT	X													
OVRLAY	X													
PUPY													X	
RMAINT								X						
SOLVE2	X													
SUMARY	X													
TIME								X			X			
USER								X						

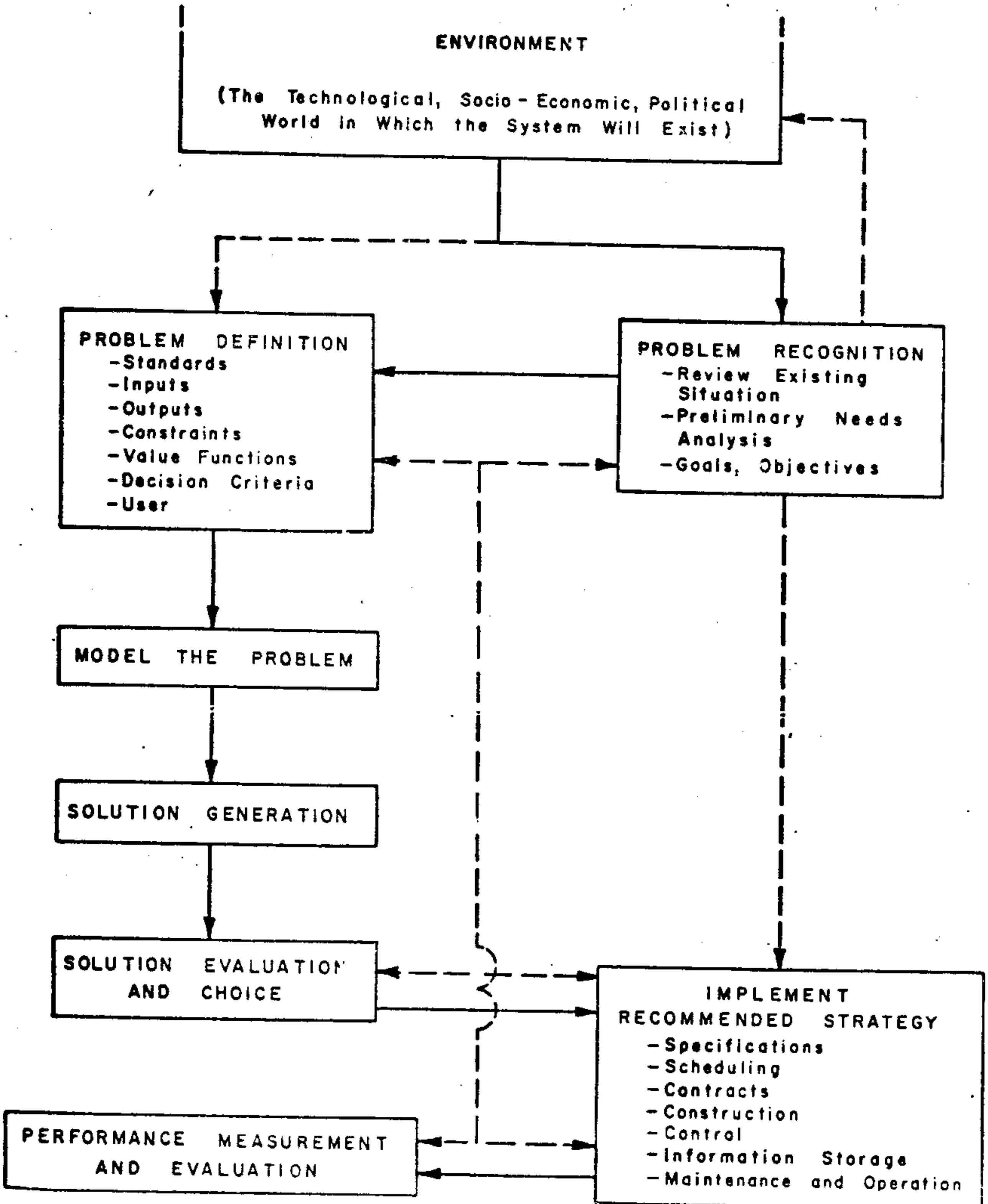


Figure 1. General representation of the systems engineering process.

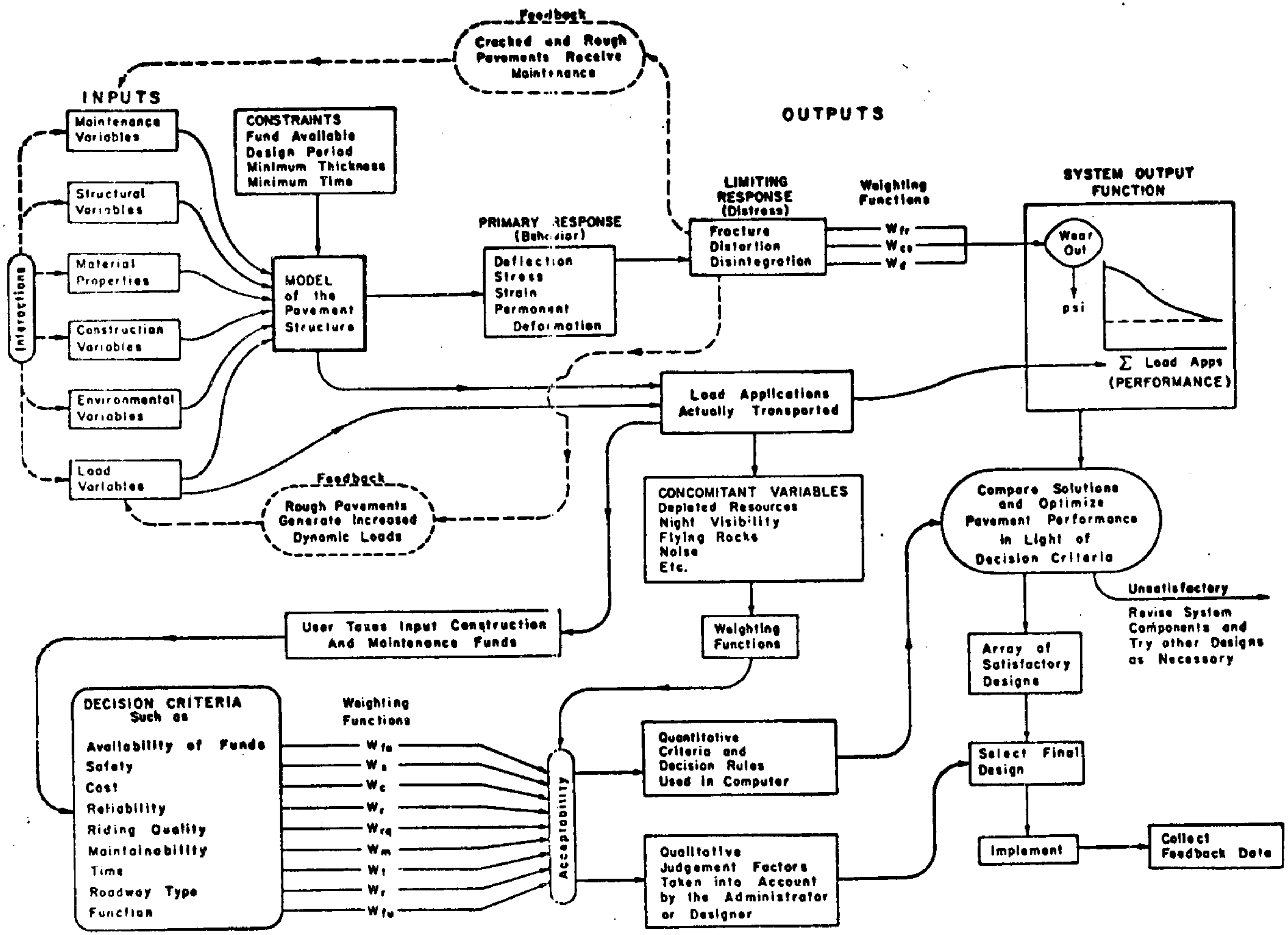


Figure 2. Block diagram of conceptual pavement system.

PROGRAM SAMPS (SYSTEMS ANALYSIS MODEL FOR PAVEMENTS) REVISED 21 MAY 70
RUN TO SOLVE THE AVERAGE VALUE PROBLEM 6 JUN 70 DLP

PROB 4000 ALL VARIABLES AT ENGINEERING AVERAGE

THE CONSTRUCTION MATERIALS UNDER CONSIDERATION ARE

LAYER	CODE	MATERIALS NAME	COST PER CY	STR. COEFF.	MIN. DEPTH	MAX. DEPTH	SALVAGE PCT.	SOIL SUPPORT
1	A	ASPHALTIC CONCRETE	10.00	.44	6.00	8.00	50.00	0.00
2	B	CRUSHED STONE	5.00	.14	5.00	8.00	50.00	9.20
3	C	GRAVEL	2.00	.11	5.00	10.00	50.00	6.90
		SUBGRADE	-0.00	-0.00	-0.00	-0.00	-0.00	4.25

PROGRAM CONTROL AND MISCELLANEOUS VARIABLES

NMB - THE NUMBER OF OUTPUT PAGES FOR THE SUMMARY TABLE (10 DESIGNS/PAGE).	3
NM - THE TOTAL NUMBER OF MATERIALS AVAILABLE. EXCLUDING SUBGRADE.	3
CL - THE LENGTH OF THE ANALYSIS PERIOD (YEARS).	20
ALW - THE WIDTH OF EACH LANE (FEET).	12
RATE - THE INTEREST RATE OR TIME VALUE OF MONEY (PERCENT)	5.0

ENVIRONMENTAL AND SERVICEABILITY VARIABLES

R - REGIONAL FACTOR.	1.7
PSI - THE SERVICEABILITY INDEX OF THE INITIAL STRUCTURE.	4.2
PI - THE SERVICEABILITY INDEX OF AN OVERLAY.	4.2
P2 - THE MINIMUM ALLOWED VALUE OF THE SERVICEABILITY INDEX. (POINT AT WHICH AN OVERLAY MUST BE APPLIED).	2.5
P2P - THE LOWER BOUND ON THE SERVICEABILITY INDEX WHICH WOULD BE ACHIEVED IN INFINITE TIME WITH NO TRAFFIC	1.5
NONE - THE RATE AT WHICH NON-TRAFFIC FACTORS REDUCE THE SERVICEABILITY INDEX.	.120

LOAD AND TRAFFIC VARIABLES

RO - THE ONE-DIRECTION AVERAGE DAILY TRAFFIC AT THE BEGINNING OF THE ANALYSIS PERIOD	10000
RC - THE ONE-DIRECTION AVERAGE DAILY TRAFFIC AT THE END OF ANALYSIS PERIOD.	20000
XNC - THE ONE-DIRECTION ACCUMULATED NUMBER OF EQUIVALENT 18-KIP AXLES DURING THE ANALYSIS PERIOD.	5000000
PROP - THE PERCENT OF ADT WHICH WILL PASS THROUGH THE OVERLAY ZONE DURING EACH HOUR WHILE OVERLAYING IS TAKING PLACE	6.0
ITYPE - THE TYPE OF ROAD UNDER CONSTRUCTION (1-RURAL, 2-URBAN).	1

CONSTRAINT VARIABLES

XTTU - THE MINIMUM ALLOWED TIME TO THE FIRST OVERLAY	2.0
XTOU - THE MINIMUM ALLOWED TIME BETWEEN OVERLAYS.	3.0
CMAA - THE MAXIMUM FUNDS AVAILABLE FOR INITIAL CONSTRUCTION	5.00
ICMAX - THE MAXIMUM ALLOWABLE TOTAL THICKNESS OF INITIAL CONSTRUCTION	32.0
OVMIN - THE MINIMUM THICKNESS OF AN INDIVIDUAL OVERLAY.	.50
OVMAX - THE ACCUMULATED MAXIMUM THICKNESS OF ALL OVERLAYS	2.5

TRAFFIC DELAY VARIABLES ASSOCIATED WITH OVERLAY AND ROAD GEOMETRICS

ACPR - ASPHALTIC CONCRETE PRODUCTION RATE (TONS/HOUR).	75.0
ACCO - ASPHALTIC CONCRETE COMPACTED DENSITY (TONS/COMPACTED CY).	1.80
XLSO - THE DISTANCE IN WHICH TRAFFIC IS SLOWED IN THE OVERLAY DIRECTION	.60
XLSN - THE DISTANCE OVER WHICH TRAFFIC IS SLOWED IN THE NON-OVERLAY DIRECTION	.60
XLSD - THE DISTANCE AROUND THE OVERLAY ZONE (MILES)	0.00
HPD - THE NUMBER OF HOURS/DAY OVERLAY CONSTRUCTION TAKES PLACE	8.0

TRAFFIC DELAY VARIABLES ASSOCIATED WITH TRAFFIC SPEEDS AND DELAYS

THE PERCENT OF VEHICLES THAT WILL BE STOPPED BECAUSE OF THE MOVEMENT OF PERSONNEL OR EQUIPMENT.	
PPO2 - IN THE OVERLAY DIRECTION	5.00
PPN2 - IN THE NON-OVERLAY DIRECTION.	5.00
THE AVERAGE DELAY PER VEHICLE STOPPED BECAUSE OF THE MOVEMENT OF PERSONNEL AND EQUIPMENT.	
DDO2 - IN THE OVERLAY DIRECTION (HOURS)	.150
DDN2 - IN THE NON-OVERLAY DIRECTION (HOURS)	.150
AAS - THE AVERAGE APPROACH SPEED TO THE OVERLAY AREA.	50
THE AVERAGE SPEED THROUGH THE OVERLAY AREA	
ASO - IN THE OVERLAY DIRECTION (MPH).	30
ASN - IN THE NON-OVERLAY DIRECTION (MPH).	50
MODEL - THE TRAFFIC HANDLING MODEL USED.	3

MAINTENANCE VARIABLES

X2-THE NUMBER OF DAYS PER YEAR THAT THE TEMPERATURE REMAINS BELOW 32F.	60
CLW - THE COMPOSITE LABOR WAGE	2.05
CERR - THE COMPOSITE EQUIPMENT RENTAL RATE.	2.50
CMAT - THE RELATIVE MATERIAL COST (1.00) IS AVERAGE).	1.00

Figure 3. Typical computer listing of input data, SAMPS.

PROB 000 ALL VARIABLES AT ENGINEERING AVERAGE

SUMMARY OF THE BEST DESIGN STATEGIES
IN ORDER OF INCREASING TOTAL COST

	1	2	3	4	5	6	7	8	9	10
MATERIAL ARRANGEMENT	A	A	A	A	ABC	ABC	ABC	ABC	ABC	AB
INIT. CONST. COST	2.153	2.222	2.014	2.083	2.639	2.917	2.917	2.708	2.708	2.639
OVERLAY CONST.COST	1.127	1.092	1.309	1.262	.825	.558	.570	.802	.802	.863
USER COST	.271	.252	.303	.297	.203	.130	.138	.201	.202	.208
ROUTINE MAINT.COST	.588	.633	.594	.592	.624	.710	.714	.631	.633	.627
SALVAGE VALUE	-.510	-.550	-.510	-.523	-.576	-.602	-.602	-.589	-.589	-.576
TOTAL COST	3.628	3.650	3.710	3.710	3.715	3.719	3.737	3.753	3.756	3.761
NUMBER OF LAYERS	1	1	1	1	3	3	3	3	3	2
LAYER DEPTH(INCHES)										
D(1)	7.75A	8.00A	7.25A	7.50A	6.00A	6.00A	6.25A	6.25A	6.00A	7.00A
D(2)					6.00B	5.00B	5.00B	5.00B	5.00B	5.00B
D(3)					6.00C	10.00C	8.75C	5.00C	8.25C	
NO. OF PERF. PERIODS	5	4	5	5	4	3	3	4	4	4
PERF. TIME(YEARS)										
1(1)	2.563	2.813	2.031	2.281	3.688	4.781	4.688	3.806	3.938	3.438
1(2)	5.797	7.734	5.266	5.844	8.469	11.625	11.344	9.063	9.234	7.844
1(3)	9.828	14.016	9.297	10.250	14.516	20.531	20.062	15.672	15.891	13.375
1(4)	14.750	21.750	14.219	15.594	21.875			23.641	24.047	20.031
1(5)	20.656		20.125	22.062						
OVERLAY POLICY(INCH) (INCLUDING LEVEL-UP)										
0(1)	1.5	2.5	2.0	2.0	1.5	1.5	1.5	1.5	1.5	1.5
0(2)	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
0(3)	1.5	1.5	1.5	1.5	1.5			1.5	1.5	1.5
0(4)	1.5		1.5	1.5						

Figure 4. Typical computer output of SAMP5.

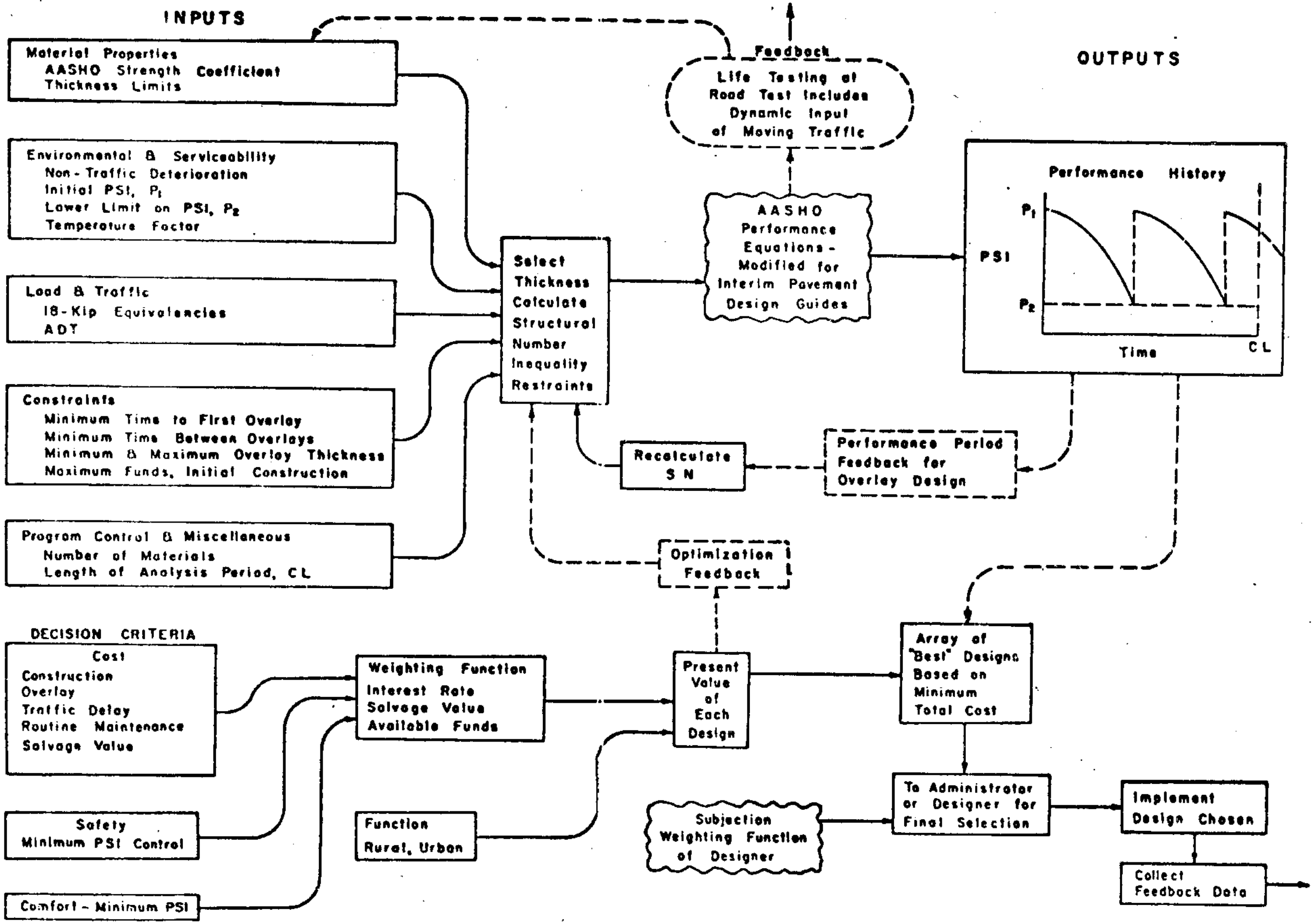


Figure 5. Block diagram of SAMP pavement system.

```
SAMP6 RUN='SAMP6 EXAMPLE PROBLEM  4 LANE INTERSTATE HIGHWAY (NO SWELLING CLAY)          PAGE  1
PROB=    ='1  MINIMUM TIME TO OVERLAY OF TWO YEARS.

INPUT DATA

     PROGRAM CONTROL AND MISCELLANEOUS VARIABLES
          NPG-THE NUMBER OF OUTPUT PAGES FOR THE SUMMARY TABLE(10 DESIGNS/PAGE).                    3
          NL-THE NUMBER OF LANES ON THE HIGHWAY (BOTH DIRECTIONS).                                  4
          CL-THE LENGTH OF THE ANALYSIS PERIOD(YEARS).                                            20.
          XLWFT-THE WIDTH OF EACH LANE (FEET).                                                    12.
          PCTRAT-THE INTEREST RATE OR TIME VALUE OF MONEY(PERCENT).                              5.00
          UPLVL-THE LEVEL-UP THICKNESS REQUIRED PER OVERLAY(INCHES).                              0.5
          WSPR-WEARING SURFACE PRODUCTION RATE(TONS/HOUR).                                       75.0

     ENVIRONMENTAL AND SERVICEABILITY VARIABLES
          R-REGIONAL FACTOR.                                                                      1.5
          PSI-THE SERVICEABILITY INDEX OF THE INITIAL STRUCTURE.                                  4.2
          P1-THE SERVICEABILITY INDEX OF AN OVERLAY.                                              4.5
          P2-THE MINIMUM ALLOWED VALUE OF THE SERVICEABILITY INDEX,                               2.0
               AT WHICH AN OVERLAY WILL BE APPLIED.
          SACT-PROPORTION OF THE PROJECT'S LENGTH LIKELY  TO SWELL                                0.0
          SRISE-VERTICAL DISTANCE THE SURFACE OF A CLAY LAYER CAN RISE(INCHES)                    0.0
          SRATE-CALCULATES HOW FAST SWELLING OCCURS                                               0.0

     LOAD AND TRAFFIC VARIABLES
          RO-THE ONE-DIRECTION AVERAGE DAILY TRAFFIC AT THE START OF THE ANALYSIS PERIOD.       5635.
          RC-THE ONE-DIRECTION AVERAGE DAILY TRAFFIC AT THE END OF ANALYSIS PERIOD.             8635.
          XNC-THE ONE-DIRECTION ACCUMULATED NUMBER OF EQUIVALENT 18-KIP AXLES DURING         6700000.
               THE ANALYSIS PERIOD.
          PROPCT-THE PERCENT OF ADT WHICH WILL PASS THROUGH THE OVERLAY ZONE DURING               5.5
               EACH HOUR WHILE OVERLAYING IS TAKING PLACE.
          ITYPE-THE TYPE OF ROAD UNDER CONSTRUCTION(1-RURAL,2-URBAN)..                              2
          COEFVR-COEFFICIENT OF VARIATION.                                                        0.0
          MCONF-CONFIDENCE LEVEL INDICATOR.                                                         3

     CONSTRAINT VARIABLES
          XTTO-THE MINIMUM ALLOWED TIME TO THE FIRST OVERLAY.                                     2.0
          XTBO-THE MINIMUM ALLOWED TIME BETWEEN OVERLAYS.                                         2.0
          CMAX-THE MAXIMUM FUNDS AVAILABLE FOR INITIAL CONSTRUCTION.                            10.00
          TMAXIN-THE MAXIMUM ALLOWABLE TOTAL THICKNESS OF INITIAL CONSTRUCTION(INCHES).         32.00
          TMOVIN-THE ACCUMULATED THICKNESS MAXIMUM OF ALL OVERLAYS  (INCHES),                   10.00
              (EXCLUDING WEAR-COAT AND LEVEL-UP).
          UPGCST-COST/CU. YD. TO UPGRADE AFTER AN OVERLAY.                                        0.0
          WIDUPG-WIDTH OF PAVEMENT & SHOULDERS TO BE UPGRADED(FEET).                              0.0

     TRAFFIC DELAY VARIABLES ASSOCIATED WITH OVERLAY AND ROAD GEOMETRICS
          ACPR-ASPHALTIC CONCRETE PRODUCTION RATE(TONS/HOUR).                                    75.0
          ACCD-ASPHALTIC CONCRETE COMPACTED DENSITY(TONS/COMPACTED CY)                           1.80
          XLSO-THE DISTANCE OVER WHICH TRAFFIC IS SLOWED IN THE OVERLAY DIRECTION.               0.50
          XLSN-THE DISTANCE OVER WHICH TRAFFIC IS SLOWED IN THE NON-OVERLAY DIRECTION.           0.50
          XLSO-THE DISTANCE AROUND THE OVERLAY ZONE(MILES).                                       0.0
          HPD-THE NUMBER OF HOURS/DAY OVERLAY CONSTRUCTION TAKES PLACE.                           8.0
          NLRO-THE NUMBER OF LANES IN THE RESTRICTED ZONE IN THE OVERLAY DIRECTION.                 1
          NLRN-THE NUMBER OF LANES IN THE RESTRICTED ZONE IN THE NON-OVERLAY DIRECTION.             2


SAMP6 RUN='SAMP6 EXAMPLE PROBLEM  4 LANE INTERSTATE HIGHWAY (NO SWELLING CLAY)          PAGE  2
PROB=    ='1  MINIMUM TIME TO OVERLAY OF TWO YEARS.

     TRAFFIC DELAY VARIABLES ASSOCIATED WITH TRAFFIC SPEEDS AND DELAYS
              THE PERCENT OF VEHICLES STOPPED DUE TO MOVEMENT OF PERSONNEL OR EQUIPMENT.
          PPO2-IN THE OVERLAY DIRECTION.                                                         5.00
          PPN2-IN THE NON-OVERLAY DIRECTION.                                                      0.0
              THE AVERAGE DELAY PER VEHICLE STOPPED DUE TO MOVEMENT OF PERSONNEL & EQUIP.
          DO2 -IN THE OVERLAY DIRECTION(HOURS).                                                 0.150
          DN2 -IN THE NON-OVERLAY DIRECTION(HOURS).                                               0.0
          AAS-THE AVERAGE APPROACH SPEED TO THE OVERLAY AREA.                                     60.
              THE AVERAGE SPEED THROUGH THE OVERLAY AREA
          ASO-IN THE OVERLAY DIRECTION(MPH).                                                      45.
          ASN-IN THE NON-OVERLAY DIRECTION(MPH).                                                  60.
          MODEL-THE TRAFFIC HANDLING MODEL USED.                                                    3

     MAINTENANCE VARIABLES
          MNTMOD-THE MAINTENANCE MODEL(EXPLICIT=1,NCHRP=2).                                         2
          CM1-INITIAL ANNUAL ROUTINE COST($/LANE MILE, MNTMOD=1).                                 0.0
          CM2-ANNUAL INCREMENTAL INCREASE IN COSTS($/LANE MILE/YR, MNTMOD=1).                     0.0
          X2-DAYS THE TEMPERATURE REMAINS BELOW 32F.(DAYS/YEAR, MNTMOD=2).                        10.
          CLW-THE COMPOSITE LABOR WAGE($/HR).                                                    2.05
          CERR-THE COMPOSITE EQUIPMENT RENTAL RATE.                                              2.50
          CMAT-THE RELATIVE MATERIAL COST(1.00 IS AVERAGE).                                      1.00
```

Figure 6. An example summary of the input data.

CROSS SECTION MODEL, COST AND SHOULDER VARIABLES

MOXSEC-THE CROSS SECTION MODEL USED.	1
MOCOST-THE COST MODEL USED.	1
MASPHS-ASPHALTIC SHOULDER MODEL (0 IF NOT ASPHALTIC SHOULDERS).	1
SOWID-WIDTH OF OUTSIDE SHOULDER, IN FEET	10.00
SIWID-WIDTH OF INSIDE SHOULDER, IN FEET	4.00
XOWID-CROSS SECTION WIDTH OUTSIDE OF OUTSIDE SHOULDER(FEET)	6.0
XIWID-CROSS SECTION WIDTH OUTSIDE OF INSIDE SHOULDER(FEET)	0.0

ADDITIONAL WIDTH(FEET) OF LAYERS RELATIVE TO LAYER ONE.

LAYER NO.	PAVEMENT-LAYERS OUTSIDE	INSIDE	SHOULDER-LAYERS OUTSIDE	INSIDE
1	0.0	0.0	0.0	0.0
2	2.00	2.00	0.25	0.25
3	10.25	4.25		
4	10.25	4.25		

TACK, PRIME, AND BITUMINOUS VARIABLES

ACTL-TACK COAT COST($/GAL).	0.0
ALPC-PRIME COAT COST($/GAL).	0.0
ACG-BITUMINOUS MATERIAL COST($/GAL).	0.0
TLMAX-MAXIMUM LAYER DEPTH FOR NO TACK COATS, INCHES	4.00
TLINC-MAXIMUM DEPTH OF EACH LIFT ABOVE TLMAX, INCHES	1.00

SAMP6 RUN='SAMP6 EXAMPLE PROBLEM 4 LANE INTERSTATE HIGHWAY (NO SWELLING CLAY)
PROB= ='1 MINIMUM TIME TO OVERLAY OF TWO YEARS. PAGE 3

THE CONSTRUCTION MATERIALS UNDER CONSIDERATION ARE

LAYER NO.	-PAVEMENT MATERIALS- CODE	DESCRIPTION	STRENGTH COEFF.	SOIL SUPPORT	----MINIMUM---- DEPTH	$/CU.YD.	----MAXIMUM---- DEPTH	$/CU.YD.	SALVAGE VALUE	INCREMENT
-	-	NO SEP. W.S.	0.0	------	0.0	0.0	------	------	0.0	------
-	-	AC.TYPE3	0.44	------	1.00	18.00	5.00	18.00	30.00	1.00
1	A	ASPH.CONC.TYPE 3	0.44	0.0	1.50	18.00	1.50	18.00	30.00	0.50
2	3	ASPH.CONC.TYPE3	0.40	10.00	2.00	18.00	12.00	16.00	30.00	2.00
3	L	LIME STAB.S-C-G	0.11	7.80	4.00	7.00	20.00	5.00	50.00	4.00
4	M	SELECT MATERIAL	0.04	3.50	4.00	2.00	10.00	2.00	50.00	2.00
-	-	SUBGRADE	------	3.10	------	------	------	------	------	

LAYER NO.	-PAVEMENT MATERIALS- CODE	DESCRIPTION	---APPLICATION RATES---- TACK COAT	PRIME COAT	ASPHALT (LB/IN)	ASPHALT CONTENT (PCT)
-		NO SEP. W.S.	0.0	0.0	0.0	0.0
-		AC.TYPE3	0.0	0.0	0.0	0.0
1	A	ASPH.CONC.TYPE 3	0.0	0.0	0.0	0.0
2	3	ASPH.CONC.TYPE3	0.0	0.0	0.0	0.0
3	L	LIME STAB.S-C-G	0.0	0.0	0.0	0.0
4	M	SELECT MATERIAL	0.0	0.0	0.0	0.0

LAYER NO.	-SHOULDER MATERIALS-	DESCRIPTION	DEPTH	$/CU.YD.	SALVAGE VALUE	---APPLICATION RATES---- TACK COAT	PRIME COAT	ASPHALT (LB/IN)	ASPHALT CONTENT (PCT)	ADJUST. VOLUME
1	-	AC-WC-SH-MIX	1.50	18.00	30.00	0.0	0.0	0.0	0.0	0.0
2	-	SELECT MAT'L	8.00	2.00	50.00	0.0	0.0	0.0	0.0	0.0
-	-	NO FILL MAT.	------	0.0	0.0	------	------	------	------	0.0
										0.0

Figure 6. (Continued)

```
SAMP6 RUN='SAMP6 EXAMPLE PROBLEM  4 LANE INTERSTATE HIGHWAY (NO SWELLING CLAY)          PAGE  7
PROB=    ='1  MINIMUM TIME TO OVERLAY OF TWO YEARS.

DESIGN TYPE   4. A 4 LAYER DESIGN
MATERIAL ARRANGEMENT A3LM

     EXCLUDING TACK, PRIME, BITUMEN, AND THE SHOULDERS,
    THE MATERIAL LAYER COSTS/(SQ.YD.) ARE . . .

LAYER -----MATERIALS-------    -DOLLARS-PER-SQUARE-YARD-
NO.  . CODE  DESCRIPTION       MINIMUM MAXIMUM INCREMENT

 1      A  ASPH.CONC.TYPE 3     0.750   0.750
 2      3  ASPH.CONC.TYPE3      1.000   6.000
 3      L  LIME STAB.S-C-G      0.778   2.778
 4      M  SELECT MATERIAL      0.222   0.556

  4   THE OPTIMAL DESIGN FOR THE MATERIALS UNDER CONSIDERATION--
       FOR INITIAL CONSTRUCTION THE DEPTHS SHOULD BE
              A     ASPH.CONC.TYPE 3    1.50 INCHES
              3   * ASPH.CONC.TYPE3     8.00 INCHES
              L     LIME STAB.S-C-G     4.00 INCHES
              M     SELECT MATERIAL     4.00 INCHES
       THE LIFE OF THE INITIAL STRUCTURE =    8.6 YEARS     STRUCTURAL NUMBER  4.46
       THE OVERLAY SCHEDULE IS
                1.00INCH(ES) (EXCLUSIVE OF LEVEL-UP AND WEAR-COURSE) AFTER    8.6 YEARS.
       THE TOTAL LIFE =    22.6 YEARS.

       THE TOTAL COSTS PER SQ. YD. FOR THESE CONSIDERATIONS ARE
              INITIAL CONSTRUCTION COST             7.653
              TOTAL ROUTINE MAINTENANCE COST        0.453
              TOTAL OVERLAY CONSTRUCTION COST       0.781
              TOTAL USER COST DURING
                     OVERLAY CONSTRUCTION           0.028
              SALVAGE VALUE                        -1.135
              TOTAL OVERALL COST                    7.780

SAMP6 PROGRAM ACTIVITY REPORT, DESIGN TYPE A3LM
 INITIAL DESIGNS
       72 WITHIN COST AND THICKNESS CONSTRAINTS
       45 FEASIBLE TO FIRST OVERLAY
 OVERLAYS
      163 CONSIDERED
       98 FEASIBLE
       72 FEASIBLE OVERLAY POLICIES
 COMPLETE DESIGNS
       43 FEASIBLE
```

Figure 7. Example output summary of an optimum design strategy for a four-layer system.

SAMP6 RUN='SAMP6 EXAMPLE PROBLEM 4 LANE INTERSTATE HIGHWAY (NO SWELLING CLAY)
PROB= ='1 MINIMUM TIME TO OVERLAY OF TWO YEARS.

PROBLEM SUMMARY OF THE BETTER FEASIBLE DESIGNS
IN ORDER OF INCREASING TOTAL COST

	1	2	3	4	5	6	7	8	9	10
MATERIAL ARRANGEMENT	A3	A3	A3LM	A3LM	A3L	A3L	A3LM	A3L	A3LM	A3LM
INIT. CONST. COST	6.049	7.215	7.653	7.349	7.296	7.808	7.831	8.074	7.527	8.009
OVERLAY CONST. COST	2.302	0.680	0.781	1.533	1.338	0.818	0.742	0.781	1.457	0.701
USER COST	0.082	0.026	0.028	0.055	0.050	0.029	0.027	0.028	0.053	0.026
ROUTINE MAINT. COST	0.115	0.515	0.453	0.231	0.400	0.480	0.452	0.453	0.276	0.484
SALVAGE VALUE	-1.101	-0.965	-1.135	-1.300	-1.202	-1.252	-1.169	-1.390	-1.334	-1.202
TOTAL COST	7.446	7.471	7.780	7.867	7.882	7.883	7.883	7.946	7.979	8.019
NUMBER OF LAYERS	2	2	4	4	3	3	4	3	4	4
LAYER DEPTH (INCHES)										
D(1)	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50
D(2)	8.00*	10.00*	8.00*	6.00*	8.00*	6.00*	8.00*	4.00*	6.00*	8.00*
D(3)			4.00	8.00	4.00	12.00	4.00	20.00	8.00	4.00
D(4)			4.00	4.00			6.00		6.00	8.00
STRUCTURAL NUMBER	3.86	4.66	4.46	4.10	4.30	4.38	4.54	4.46	4.18	4.62
NO.OF PERF.PERIODS	4	2	2	3	3	2	2	2	3	2
PERF. TIME (YEARS)										
T(1)	3.3	11.4	8.6	4.9	6.8	7.6	9.6	8.6	5.6	10.8
T(2)	9.8	28.6	22.4	14.0	18.4	20.4	25.0	22.8	15.6	27.5
T(3)	15.5			21.7	21.6				24.1	
T(4)	20.7									
OVERLAY POLICY(INCH) EXCLUSIVE OF LEVEL-UP & WEAR-COURSE)										
D(1)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
D(2)	1.00			1.00	1.00				1.00	
D(3)	1.00									

Figure 8. Example output summary of the better over-all designs.

DESIGN SUBSYSTEMS USING MECHANISTIC ANALYSIS

Over the past thirty years there have been many efforts by individuals and organizations to establish design procedures for thickness of pavements which would be based on some rational criteria. Some of these methods are widely used, a few have even become firmly established within certain design organizations. These methods can be grouped as follows:

(1) Elasticity Methods: according to this classification consider the behavior of pavements under working conditions, when deflections, by assumption, remain proportional to applied loads. Their principal design criterion consists of limiting stresses or strains as determined by a calculation based on Theory of Elasticity to certain values established by observation to be "safe." Some methods of this group are:

a) Kansas Highway Department Method

b) Portland Cement Association Method

c) U. S. Corps of Engineers (CBR) Method

d) Shell Method

e) Texas Highway Department Method

(2) Ultimate Strength Methods: are concerned with pavement behavior at failure. Their basic design criterion is that the pavement must possess an adequate safety factor against assumed shear failure of the pavement system. This group include:

a) Early English Method

b) Yield-Line Method

(3) Semi-Empirical and Statistical Methods: are based on assembled (and in some cases statistically processed) information of conditions under which pavements of certain composition and strength have experienced performance failure. They included no theoretical considerations of pavement mechanics, yet the thicknesses are determined by an empirical "strength" test. The methods belonging to this group include:

a) The Original CBR Method

b) State of California Method

c) Canadian Department of Transport (McLeod) Method

d) AASHO Interim Guide for Flexible Pavements

e) AASHO Interim Guide for Rigid Pavements

f) Asphalt Institute Method

g) Kentucky Method.

(4) Empirical and Environmental Methods: relate the pavement thickness to some particular soil and environmental conditions. No mechanical tests, other than those needed for soil classification, are used for determining the supporting characteristics of the pavement subgrade. Two of these are:

a) Michigan Highway Department Method

b) Canadian Good Road Association Method.

Reviews of some of the methods that are widely used, especially those of Group 3 are given in this paper. Each of the reviews will include a short description of the history, principal design criteria, assumptions, determination of material properties and experience with the method, where applicable. A short bibliography will be given at the end of each method.

SHELL METHOD OF THICKNESS DESIGN FOR FLEXIBLE PAVEMENTS

History: This method was developed by engineers of the Shell Oil Company during the 1950's and early 1960's.

Principal Design Criterion:

1. The horizontal (radial) tensile strain on the underside of the asphalt-bound layer must be sufficiently small to prevent cracking in the asphalt layer.

2. The vertical compressive strain in the surface of the subgrade must be maintained sufficiently small to prevent permanent deformation of the top of the subgrade.

Assumptions:

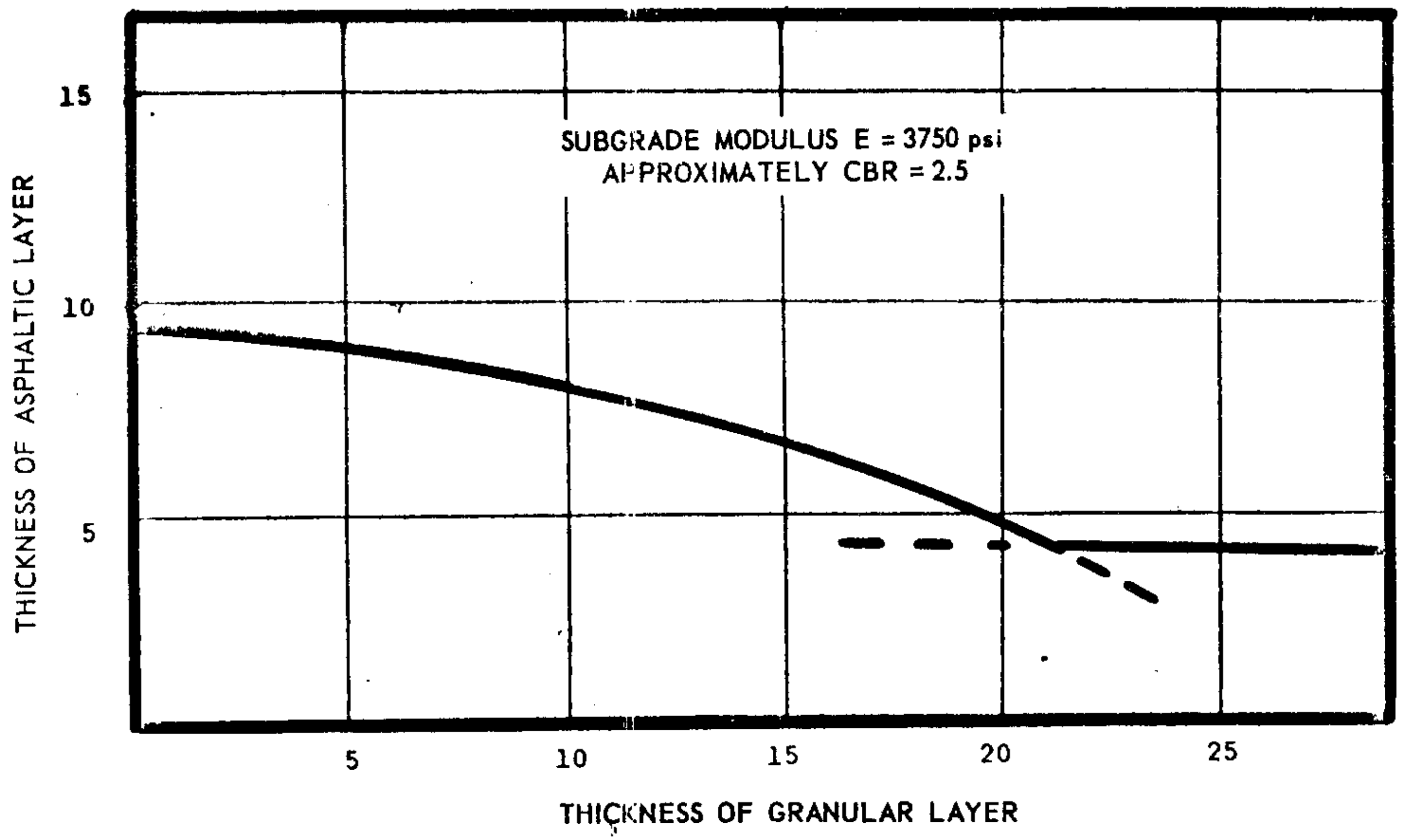
1. The pavement structure can be represented as a three-layer elastic system with no relative displacement between layers at their boundaries.

2. Subgrade strains can be determined by considering a 9,000 lb. wheel load applied uniformly over a single circular area with a six-inch radius.

3. Tensile strains in the asphalt layer are determined using a circular area with a radius of 4.2 inches subjected to a uniform contact pressure of 80 psi.

Determination of Material Properties: The only experimental determination of material properties is the CBR of the subgrade soil. This has been previously correlated to the subgrade modulus and the modulus of the aggregate base. The elastic stiffness of the asphaltic concrete is assumed at an average value.

Experience: Comparisons between thicknesses developed by this procedure and results from both sections of the AASHO Road Test still in good condition after two successive spring periods with a PSI of 2.5 or greater after 1 million load applications support the results of this design method.



SHELL THICKNESS DESIGN CURVE (10^6 APPLICATIONS)

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CBR METHOD OF THICKNESS DESIGN FOR FLEXIBLE PAVEMENTS

<u>History</u>: The California Bearing Ratio (CBR) test and design method were developed by O. J. Porter prior to 1930, while Porter was Materials Engineer for the California Highway Department. The test procedure and design method have been modified somewhat by various agencies, which employ the method. Despite this, the basic philosophies involved and the method in general remain as originally established by Porter.

<u>Principal Design Criterion</u>:

1. Materials within a pavement structure must be protected (or insulated) from surface loadings by overlying layers whose thickness can be empirically related to the strength of the material as indicated by the CBR test. Porter, originally, and many others since have established experience relations between thickness required and CBR.

2. Extensions and modifications to permit treatment of traffic volume, minimum ultimate subgrade strengths, etc., vary greatly between organizations currently using the CBR method.

<u>Assumptions</u>:

1. Surface loading is distributed to the lower layers in a pavement structure as a function of thickness alone.

2. The strength required of any layer in a pavement structure to sustain the loading delivered to it from above is directly reflected by its CBR value.

<u>Determination of Material Properties</u>: The CBR is generally determined as indicated in ASTM D-1883. No other material properties are required by the basic, unmodified CBR method.

<u>Experience</u>:

1. The CBR method of flexible pavement design is currently used by a number of state highway departments.

2. This method has been extended for use in airfield design by the U. S. Corps of Engineers and has been used satisfactorily for over 25 years for design and evaluation of military airfields throughout the world.

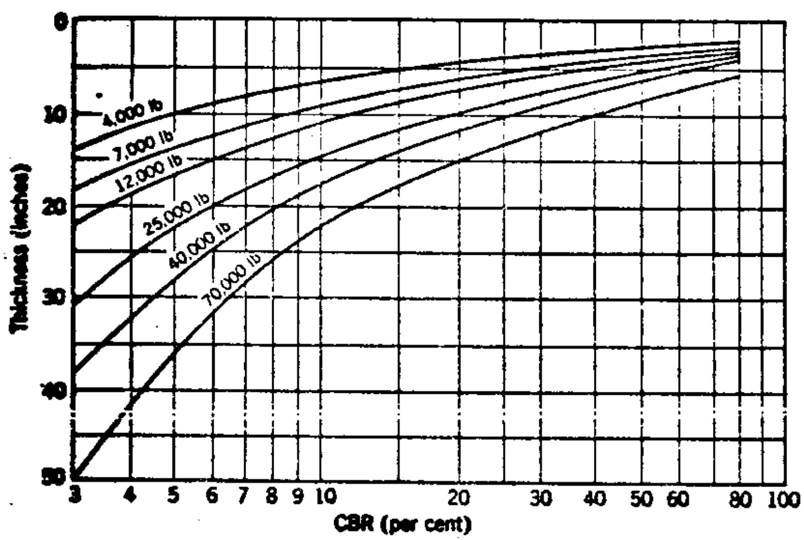

Figure Original airport design curves. (From Figure 7 of Middlebrooks and Bertram, "Development of CBR Flexible Pavement Design Methods for Airfields," a symposium *Transactions*, ASCE, 1950.)

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THE STATE OF CALIFORNIA (AND STATE OF WASHINGTON) METHOD FOR THICKNESS DESIGN OF ASPHALTIC CONCRETE PAVEMENTS

History: This method was developed by Hveem in the 1940's and has been modified on the basis of data from road tests as well as from in-service highways in California.

Principal Design Criterion: Pavement thickness in this empirical method is determined by the requirement that permanent deformation in each layer of the pavement system be prevented. The thickness desired to accomplish this is considered to be a function of the traffic, tensile properties of the paving materials, and the shear strength characteristics of the paving materials as measured by the stabilometer tests.

Assumptions:

This method is largely empirical, and therefore is based on the general assumption that the relationships developed apply under all conditions in which they will be used. In addition the following assumptions are made:

1. The effect of traffic repetitions and vehicle weight can be expressed by a "Traffic Index" which is presumed to be a measure of the number of repetitions of a 5000 pound equivalent wheel load during the design life.

2. The stabilometer tests can provide an appropriate measure of the strength characteristics of the paving materials for use in design.

3. There is an equivalency between various thicknesses of different materials in the pavement system. This equivalency is a function of the tensile properties of the material, and can be described in terms of an empirically-determined "gravel equivalent factor" which represents the thickness of a given material equivalent to a unit thickness of gravel subbase.

Determination of Material Properties: The material properties used to select a pavement thickness by this method are:

1. "R"-Value as measured by the stabilometer. Normally a particular material is prepared at a series of water contents and dry densities.

2. The exudation pressure. This is the pressure required to force water from a sample of the appropriate material which has been compacted in a mold by kneading methods.

3. Tensile strength of paving components as measured by the cohesiometer test.

4. Expansion pressure of subgrade materials.

Experience: This procedure has been used successfully by the California Division of Highways in a number of modified forms since the early 1940's. It is also used by a number of other Western states.

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CANADIAN DEPARTMENT OF TRANSPORT METHOD FOR THICKNESS DESIGN OF FLEXIBLE PAVEMENTS

History: This empirical method is based on conclusions drawn from evaluation by N. W. McLeod of the performance of Canadian airport pavements.

Principal Design Criterion: The surface deflection of a pavement under repeated load is the criterion for determining the adequacy of the structural design.

Assumptions:

1. The load carrying capacity of a given pavement can be determined from the results of a repeated plate loading test, conducted at the surface of the pavement. The contact area of the plate must be the same as the contact area of the tire considered to apply pressure to the pavement.

2. The effect of one pavement material, i.e. asphaltic concrete or granular base course, relative to that of another material in the pavement system is proportional to the ratio of their thicknesses. Thus the concept of equivalency of a thickness of one material to a different thickness of another material is assumed.

3. The pavement deflection arises from the deflection of the subgrade under load. This load carrying capacity of the subgrade can be measured by a repeated plate loading test.

Determination of Material Properties:

1. The subgrade strength is determined by repetitive static plate tests according to ASTM Designation: D 1195. It is expressed in terms of the load carried at the surface of the subgrade by a 30-in. diameter rigid plate at 0.5-in. deflection after 10 repetitions of loading. Since load testing is normally conducted in summer, the bearing value obtained thus are reduced to allow, in part, for the reduction in bearing capacity in the spring.

   If no test of any kind can be obtained, the load carrying capacity of the subgrade may be estimated on the basis of soil classification, moisture, drainage conditions, etc.

2. Standard densely graded, hot mix asphaltic concrete mixture made up of two layers of different gradation is used for the wearing course. The mixture is designed according to the Marshall Method (ASTM Designation: D 1559). The penetration grade of the bitumen in the mixture is based on the freezing index of the location and the particular use of the mixture.

3. Standard soil-aggregate materials are used for the base course and the subbase. The quality of these materials is controlled by Specification requirements.

Experience: According to Dr. G. Y. Sebastyan, Chief, Engineering Design Division of the Construction Engineering and Architectural Branch of the Canadian Department of Transport, the following problems have been encountered in using the method:

1. The correlation determines the overall pavement thickness requirement and the design does not provide for the stability of the various pavement structural components.

2. For a given subgrade strength and applied load the total pavement thickness determined by this method will be mainly related to the value "K" or the size of plate or contact area used in the design, which give rise to some inconsistency. This was one of the reasons why the Department standardized on a 30-in. diameter plate in pavement design and evaluation work.

3. The original study indicated a constant "K" value for given diameter plate. As "K" is an inverse measure of pavement strength increase per inch thickness, its value, within certain limits should be a function of the type of the material incorporated into the pavement structure.

4. "K" should also be a function of the thickness of the various pavement components. This matter has been investigated by Dr. McLeod and indirectly by Dr. Sebastyan.

5. The correlation between the pavement secant modulus of deformation and pavement performance is based on subjective knowledge and experience.

6. The design equation gives some difference in results if the contact area of the applied load is converted to an equivalent 30-in. plate on the pavement surface and then compared with a subgrade strength value determined on a 30-in. diameter plate and alternatively, if the subgrade strength is given on an equivalent plate size basis and compared with the actual contact area on the surface.

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AASHO INTERIM GUIDE FOR THE DESIGN OF FLEXIBLE PAVEMENTS

History: This method was developed from results of the AASHO Road Test by the AASHO Operating Committee on Design and released for study and trial use in 1961.

Principal Design Criterion: The adequacy of the structural design is measured by the change in the Present Serviceability Index (PSI) of the pavement. The PSI is defined as the momentary ability of a pavement to serve traffic. The change in PSI is a function of maximum axle load, number of axles, number of load repetitions, seasonal effects, thickness of pavement components and the subgrade soil support.

Assumptions:

1. The significant relationships found between the number of repetitions of specified axle loads, the thicknesses of surface, base and subbase, and the basement soil used on the Road Test are valid for all soil types. This assumption permits the establishment of a soil support scale.

2. Each axle load applied to a pavement structure in a mixed traffic stream has the same relationship to pavement performance as was found in the Road Test for pavements carrying axle loads of a fixed magnitude. This assumption leads to the development of equivalence factors and the expression of traffic loadings in terms of a common denominator.

3. An environmental factor may be introduced into the design analysis on the basis of a summation of the seasonal weighting factors used to weight the axle load applications on the Road Test.

ASPHALT INSTITUTE METHOD FOR THICKNESS DESIGN OF ASPHALT PAVEMENTS

History: This method was developed from the statistical analysis of data from the AASHO Road Test. Information from the WASHO Road Test, from British Test Roads, older editions of the Asphalt Institute manual, and state highway and other existing design procedures was incorporated. The present version was published in 1963.

Principal Design Criterion: The adequacy of the structural design is measured by the change in the Present Serviceability Index (PSI) of the pavement. The PSI is defined as the momentary ability of the pavement to serve traffic. The change in PSI is a function of the number and rate of all axle and wheel loads, strength of the subgrade soil in its critical moisture condition and strength, or relative strengths of the surface, base and subbase courses.

Assumptions:

1. The number of applications of various wheel loads can be expressed in terms of an equivalent number of 18,000 pound single-axle load applications. This assumption leads to the development of equivalence factors and the expression of traffic loadings in terms of a common denominator.

2. The effect of one pavement material, i.e., asphaltic concrete or granular base course, relative to that of another material in the pavement system is proportional to the ratio of their thicknesses. Thus the concept of equivalency of a thickness of one material to a different thickness of another material is assumed.

3. The relationships found between the equivalent pavement thickness, the equivalent number of 18 kip axle loads and the soil support value for the AASHO Road Test data are valid for all soil types.

4. The soil support value can be measured by either the California Bearing Ratio (CBR) or the Stabilometer R-value.

Determination of Material Properties: The soil support value is determined by either the California Bearing Ratio test or the Stabilometer test. No determination of properties of the other pavement components is made. However it is assumed that all such components exhibit at least the minimum quality specified in terms of gradation, CBR, Marshall stability and other standard tests normally used in highway work.

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AASHO INTERIM GUIDE FOR THE DESIGN OF RIGID PAVEMENTS

History: This method was developed from results of the AASHO Road Test by the AASHO operating committee on design and released for study and trial use in 1961.

Principal Design Criterion: The adequacy of the structural design is measured by the change in the Present Serviceability Index (PSI) of the pavement. The PSI is defined as the momentary ability of a pavement to serve traffic. The change in PSI is considered to depend upon the maximum tensile stress in the concrete relative to the tensile strength. This stress is a function of maximum axle load, number of axles, number of load repetitions, seasonal effects, thickness, stiffness, and strength of the concrete, and the subgrade soil support.

Assumptions:

1. The significant relationships found between the number of repetitions of specified axle loads, rigid pavement thickness, characteristics of the pavement and subgrade on the AASHO Road Test are valid for all soil types.

2. Each axle load applied to a pavement structure in a mixed traffic stream has the same relationship to pavement performance as was found in the AASHO road test for pavements carrying axle loads of a fixed magnitude. This assumption leads to the development of equivalence factors and the expression of traffic loadings in terms of a common denominator.

3. An environmental factor may be introduced into the design analysis on the basis of a summation of the seasonal weighting factors used to weight the axle load applications on the AASHO road test.

4. The capacity of the subgrade to support traffic loadings can be measured by the modulus of subgrade reaction.

5. The maximum tensile stress in the concrete can be predicted from the Spangler equation of corner load stresses.

Determination of Material Properties: The material parameters required are the modulus of rupture of the concrete and the modulus of subgrade reaction of the subgrade. The modulus of rupture of the concrete is determined from a compression test on 28-day old concrete using the test procedure specified in AASHO Designation T-97. The modulus of subgrade reaction is determined from a plate loading test conducted in accordance with ASTM Designation D 1196-57, using a 30-inch diameter plate. This modulus may also be estimated in such cases as this is warranted by experience.

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THE KENTUCKY METHOD

History: This method was developed in 1958 and presented by Havens, Deen and Southgate to fit conditions in Kentucky.

Principal Design Criterion:

The subject design, like other elastic layered design procedures, considers both permanent deformation (rutting) as well as fatigue cracking of the asphalt-bound layer as the two most significant failure mechanisms. The information necessary to solve the pavement thickness solution is the CBR of the subgrade, design number of equivalent axle load and the desired thickness percentage of A.C. in the structure.

Assumptions:

1. The method is based on CBR and equivalent wheel load.
2. Use of limiting tensile strain criteria for fatigue cracking.
3. Development of a F value for the granular base modulus to account for the known dependency of the value upon the underlying subgrade modulus.
4. E_1 range from 150,000 to 1,800,000 psi

   $E_2 = F \times CBR \times 1500$

   $E_3 = CBR \times 1500$
5. Poisson;s ratio of layer 1 = 0.40

   layer 2 = 0.40

   layer 3 = 0.45
6. Tire pressure = 80 psi
7. $F = 1$ when $E_1 = E_2 = E_3$
8. Procedure is based on arriving at design thickness for each of the two distress modes.

Description of Material Properties

Soil support value is determined by the California Bearing Ratio Test.

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MECHANISTIC STRUCTURAL SUBSYSTEM PROGRAMS

VESYS IIM

PAVEMENT LAYER THICKNESSES
SURFACE - ASPHALT CONCRETE
BASE/SUBBASE - ASPHALT TREATED
- CEMENT TREATED
- UNTREATED

TRAFFIC
ESTIMATED TRUCK VOLUME AND W-4 DISTRIBUTIONS
ANNUAL CHANGES
VEHICLE SPEEDS

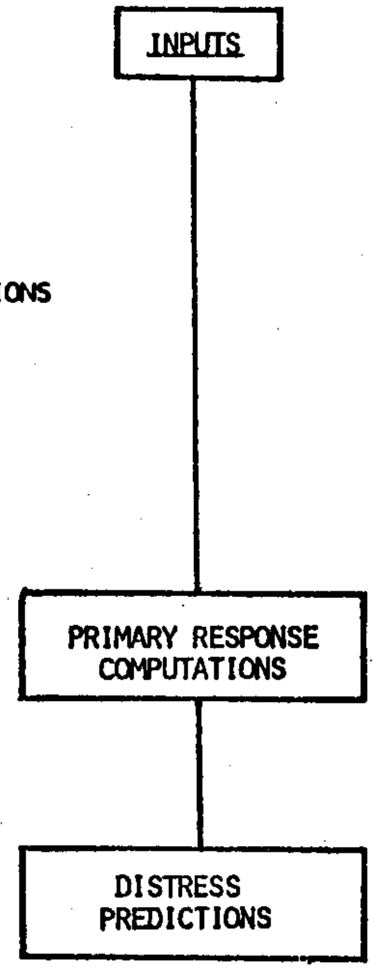
MATERIAL CHARACTERISTICS
LINEAR ELASTIC & VISCOELASTIC (SURFACE, BASE/SUBBASE, & SUBGRADE)
SEASONAL AND ENVIRONMENTAL VARIATIONS
FATIGUE PROPERTIES
PERMANENT DEFORMATION PROPERTIES

MODELS
THREE LAYER VISCOELASTIC
PROBABILISTIC
TIME & TEMPERATURE SENSITIVITY

FATIGUE CRACKING
LABORATORY TEST CALIBRATION

RUTTING
LABORATORY TEST CALIBRATION

ROUGHNESS
F (MATERIAL VARIABILITY)



PDMAP

PAVEMENT LAYER THICKNESSES
SURFACE - ASPHALT CONCRETE
BASE/SUBBASE - ASPHALT TREATED
- CEMENT TREATED
- UNTREATED

TRAFFIC
AASHO EQUIVALENT 18 KIP LOADS
ANNUAL CHANGES

MATERIAL CHARACTERISTICS
LINEAR ELASTIC (SURFACE, BASE/SUBBASE, & SUBGRADE)
SEASONAL AND ENVIRONMENTAL VARIATIONS
ANNUAL CHANGES
FATIGUE PROPERTIES

MODELS
FIVE LAYER ELASTIC
SUPERPOSITION
TEMPERATURE & STRESS SENSITIVITY

FATIGUE CRACKING
AASHO ROAD TEST CALIBRATION

RUTTING
AASHO ROAD TEST CALIBRATION

LOW TEMPERATURE CRACKING
ST. ANNE ROAD TEST VERIFICATION

FIGURE 2

AN APPLICATION: "PAVEMENT RESPONSE AND EQUIVALENCIES FOR VARIOUS TRUCK AXLE-TIRE CONFIGURATIONS"

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Pavement Response and Equivalencies for Various Truck Axle-Tire Configurations

ABSTRACT

Many changes in allowable loading and operating procedure for trucks are under consideration in Washington and other states. For example, dual tires versus single "flotation" tires for heavy truck loads may have varying damaging effects on pavements. Further, at least for asphalt pavements, the time of year and vehicle speed may also influence the analysis for special heavy load permits. This paper is a brief attempt to consider some of these variables on a relative basis.

This paper is intended to be a limited state-of-the-art approach to answer several pertinent questions from a theoretical study based on hypothetical pavements and loads, but based on reasonable material characteristics and pavement behavior from previous research. The computer program was used to compute structural behavior. By using known fatigue and failure design curves, maximum allowable numbers of load applications were determined. The results are a series of relationships based on pavement life which can be used to determine any number of "equivalencies." These equivalencies can be used to compare the relative destructive effects of various sizes of single and dual tires, axle loads, pavement thicknesses, speed and temperatures.

The general nature of these relationships provide a wide range of conditions for comparison on a relative basis. Within reason, interpolation is valid. One must keep firmly in mind, however, the fact that these relationships are for assumed conditions (although reasonable) and do not represent actual pavements.

INTRODUCTION

There are indications that many trucks now have front axle loads approaching the maximum allowable (18,000 lbs. in Washington) for single axles. The fact that these are on single tires increases the potential for pavement damage. In addition, many changes in allowable loading are under consideration in other states. For example, substituting single "flotation" tires for dual tires on heavy truck loads may have varying damaging effects on the pavements. Further, at least for asphalt pavements, the time of year and vehicle speed may also influence the analysis for special heavy load permits. This paper is a brief attempt to point out the relative effects of some of these variables.

Most of the past studies and literature deal only with relationships between dual tired single axle and dual tired tandem axles. Therefore, this study is to examine relative destructive effect of the single tire versus dual tires and especially between wide single tire (super single, flotation) and conventional dual tires.

Although this paper is not intended to be a "state-of-the-art" approach, it is a summary of a larger study (1) based on hypothetical pavements (with reasonable material characteristics and pavement behavior determined from previous research) and loads, to answer several pertinent questions using current technology. The results are a series of behavior-response relationships based on pavement life. One can then use them to determine any number of "equivalencies" of relating variables based on a certain set of conditions.

Basic variables considered in this study include:

1. Wheel load,
2. Tire contact pressure and width,
3. Thickness and nature of pavement layers,

4. Speed of vehicle, and
5. Pavement temperature.

A three-layer pavement structure with an asphalt concrete surface, AC, untreated aggregate base, UTB, and natural soil subgrade were selected for this study. Multilayered elastic theory is used to compute structural response.

The deflections, stresses, and strains are computed by using the Chevron n-layer computer program. The horizontal tensile strain at the bottom of the asphalt treated layer, ε_h, and the vertical compressive strain at the top of the subgrade, ε_v, are examined in determining the maximum number of load applications to failure (see Figure 1). The maximum strains, especially in the thin pavement layers, can occur directly under one of the dual wheels rather than midway between them.

The maximum number of load applications, N, are determined (a) from the criteria developed from laboratory fatigue tests on asphalt to minimize pavement cracking from repeated loading and (b) from the criteria developed from that of Shell to minimize surface rutting caused by over stressing the subgrade.

Based on the maximum number of the standard axle load applications, fatigue and rutting equivalencies are established. These equivalencies can be used to compare the destructive effects of various sizes of single and dual tires, and various axle loads.

The effects of changes in the level of temperature and variation in vehicle speeds on the fatigue in terms of the maximum number of load applications are also examined.

BACKGROUND INFORMATION

As noted by a number of researchers, the assumption of the pavement responding as a layered elastic system appears reasonable at this time. Several computer solutions are available and CHEV 5L was selected for this project to facilitate determination of the stresses. When using such solutions for material response characteristics that depend on stress, it is necessary to use an iterative type of solution for pavement section containing a granular material.

Material Characteristics

In general, asphalt materials are visco-elastic and therefore their stress-strain characteristics are dependent on both time of loading and temperature. A number of experimental methods have been developed to describe the relationship between stress and strain for asphalt mixes (2). For this study, the resilient modulus (M_R) for typical Washington mixtures was used and was confirmed by the stiffness concepts of Van der Poel, et. al., (3,4).

For untreated granular materials and untreated materials such as the subgrade, a measure of stiffness termed the resilient modulus can be determined from:

1. The resilient modulus test using repeated load triaxial apparatus (5).
2. The California bearing ratio (CBR) test (6).
3. The repeated plate load test (7).

In this study, data from the WSU test track studies (8) and other tests of typical Washington materials (9) were used for the subgrade and base layers. Other materials and localities would require appropriate adjustments.

Distress Criteria

Fatigue has been defined as the phenomenon of fracture under repeated or fluctuating stress having a maximum value generally less than the tensile strength of the material. Stiffness plays a predominant role in determining the fatigue behavior of asphalt mixes and maximum principal strain is a major determinant of fatigue crack initiation.

In practice, payements are subjected to a range of loadings; accordingly, a cumulative damage hypothesis is required since fatigue data are usually determined from the results of simple loading tests. One of the simplest of such hypotheses is the linear summation of cycle ratios. Fatigue life prediction under compound loading becomes a determination of the time at which this sum reaches unity.

Numerous studies have been conducted to evaluate the fatigue behavior of asphalt concrete and data are available for materials similar to those used in Washington (10). These data were used as the basis for cumulative fatigue damage due to truck traffic.

The other mode of failure considered in this study is permanent deformation, more commonly called rutting. Structural failure due to rutting can occur in one or more pavement layers. For example, recent research has shown that rutting in the asphalt layers can be predicted reasonably well. This failure mode may be particularly important for thick asphalt sections in hot climates.

A somewhat broader approach to rutting failure is utilized in the Shell design method (11) as well as in this study. It appears that there exists a critical subgrade stress/strain level beyond which the rutting is extended into the subgrade soil. For example, if the vertical subgrade strain does not exceed the critical level, ruts are not formed due to subgrade failure. From experience in Washington, rutting failure is not a significant factor, but was used here to illustrate the different failure modes which the highway engineer must guard against.

Other Factors

Pavement temperatures can be computed from weather data by solving the heat conduction equation by numerical technique, such as finite-difference procedure or finite-element procedure, or by closed form techniques as presented by

Barber (12). Alternatively, a representative temperature can be estimated by one of several methods.

As indicated above, fatigue life is dependent on the asphalt stiffness and temperature changes throughout the year will affect the analysis of pavement life accordingly. Considering a limited example in this paper, reasonable pavement temperatures were assigned and computations of pavement life made to show this relative effect.

In addition to temperature, speed of the moving vehicle has often been considered to affect stresses induced in the pavement and, ultimately, the pavement life. In order to illustrate this factor, only the change in stiffness of asphalt as caused by variable rates of loading was considered. The other pavement layers were assumed to be unaffected, for simplicity.

COMPUTATIONS

Various wheel loads and tire widths to be considered as input variables in the computer analysis were suggested by the Washington State Highway Department. Although it is generally known that the tire-pavement pressure interface is somewhat complex and affected by tire design, simplifying assumptions were made to accommodate the CHEV 5L program. These include a circular contact area with tire pressure equal to the contact pressure. Various tire pressures were calculated by dividing the wheel load by a reasonable contact area.

Several thicknesses of pavement structure have been selected: 3-in., 6-in. and 9.5 in. of asphalt concrete surface on 8 in. of untreated base. The subgrade layer is assumed to be semi-infinite.

The types of material in each layer of the pavement structure selected for this study are based on the availability of the laboratory and field test data in combination with those common in Washington. They are Class "B" wearing course, untreated aggregate base and the natural undisturbed clay subgrade soil.

The CHEV 5L computer program (13) was used to calculate the stresses, strains and deflections. The effect of the dual load on any point is then determined by linear superposition of the effects of each of the loads at the point in question. The application of superposition implies linear response and thus, the utilization of this principle is an approximation of the dual wheel load.

The computational procedure includes several iterative steps. For a particular computation, these can be summarized as follows:

1. Select thickness of each layer.
2. Estimate modulus and Poisson's ratio for each layer.
3. Select wheel load and contact area (radius of circular area of contact with pavement).
4. Select points for calculation of stresses, strains and displacements. These will usually include depths ranging from the surface downward at least into the subgrade. Points are selected radially from the center of load sufficiently far to include the adjacent dual tire (if any). Preliminary calculations indicated that tires at the opposite end of the axle from those under consideration do not contribute significantly.
5. Following computer calculation, appropriate values are selected from the printout and compared to the material behavior data. If the required moduli are not within the given range, they are adjusted and the computation repeated until reasonable agreement is attained. When dual tires are utilized, the additive values are used for this comparison so that maximum values are always considered.

6. When agreement is attained, the final iteration is used as being representative of that combination of load and pavement response.
7. The above steps are repeated for each combination of load, tire width, pavement thickness, etc.

EQUIVALENCY DETERMINATION

Surface deflection is often a good indicator of pavement behavior changes, but in itself it cannot be readily related to performance over a wide range of conditions. Therefore, the main concern will be with radial tensile strain on the bottom of asphalt concrete layers as it relates to fatigue cracking or failure. In addition, vertical compressive strain on the subgrade is examined with respect to its relationships to limiting rutting in the pavement structure.

Utilizing the computed data, a series of steps were made to reduce these data to a form directly applicable to pavement life. This was a relatively straightforward but time consuming procedure to eventually arrive at the primary relationships shown in Figures 2 and 3.

Fatigue

A complete and reasonably convenient summary of all the data is shown in Figure 2. In this figure, the relative number of applications of a particular load or combination of loads can be determined. As a basis for comparison in establishing Figure 2, a "standard" condition was defined as an 18-kip axle load with 10-inch wide dual tires on a pavement with 6 inches of asphalt concrete. Thus, this point on Figure 1 has an equivalency equal to unity. With the data normalized in this manner, any two points can be compared (divided) directly using the relative equivalencies on the vertical scale.

Rutting

Similar to that for fatigue, Figure 3 has been prepared as a summary of all the combined data for rutting behavior. Use of these data are similar to that for Figure 2. Any two points can be compared in terms of their relative life to failure in terms of their ratio or equivalency.

Climate (Temperature)

For the average case, 68.5^{o}F was assumed to be the temperature for asphalt concrete at all depths. It is known, however, that a range of temperatures is encountered such as during the summer and winter months. A profile of temperature within the asphalt concrete for these pavements was estimated from weather data.

Because of the excessive computations and analysis required, the effect of temperature on pavement behavior is limited to the case for 10-inch tires, 18-kip axles load, and for the usual range in asphalt concrete thickness, appropriately adjusted for stiffness by sublayers. Using the same fatigue criteria as before, the number of load applications to failure were determined and plotted in the format shown in Figure 4. Remembering that these data are for 18-kip axle loads only, they can be extrapolated to include other axle loads by utilizing the linear nature of curves in Figure 2.

Vehicle Speed

Speed analysis has also been limited to the standard 18-kip axle load with 10-inch wide tires. Using the 10-inch diameter contact area between tire and pavement as the contributing loaded area, the wheel was assumed to be rolling at a range of speeds and these were converted to load duration. These loading times were then used to determine stiffness of the asphalt concrete based on the principles of Van der Poel and calculated for a particular mix as by Monismith, *et al*. (14) and from data on Washington mixtures (15).

Using fatigue data, the relative effect of speed on the number of applications of load to failure is shown in Figure 5. Although the basic equivalency relationships shown earlier in Figures 2 and 3 did not indicate a speed, most of the stiffness data were developed for testing load duration of about 0.5 sec., i.e., about 10 mph. The user of these data should be cautioned, however, that relative values only should be compared and not actual load applications to failure, for example.

SUGGESTED USE AND APPLICATIONS

Appropriate utilization and recognition of the limiting factors in this study are very important. The user must realize that comparisons among the variables considered are only relative and should not be used for actual pavement life predictions, for example. This approach is predicated on the fact that computed data are based on hypothetical pavements, although they are reasonable approximations of typical pavements constructed in Washington.

Using the key relationships developed herein, Figures 2 through 5 can be used to determine a wide range of equivalencies as illustrated in the form of examples or typical situations.

Example 1

Compare the relative pavement fatigue life expectancy of a 3-inch asphalt pavement when subjected to an 18-kip axle load with 10-inch dual tires and 18.5-inch single flotation tires.

Solution

From Figure 2, the relative life for the single and dual cases are 250×10^{-3} and 180×10^{-3}, respectively. Therefore the equivalency of these two are:

$$\frac{250 \times 10^{-3}}{180 \times 10^{-3}} = 1.38$$

i.e., the single tire would be 38% more damaging in terms of fatigue, or conversely, the pavement will last 38% longer under the dual tires.

One should note that these equivalencies are compared for conditions that are all constant except for those being compared.

Example 2

Determine the equivalency for the same loads as in Example 1, but in terms of rutting distress.

Solution

From Figure 3, for 3-inch asphalt concrete, for dual tires, $N = 32 \times 10^{-3}$ and the single $N = 14 \times 10^{-3}$. The equivalency would then be: $\frac{32 \times 10^{-3}}{14 \times 10^{-3}} = 2.3$ for rutting damage, indicating that the pavement will last 2.3 times longer under dual tires.

Assuming that the pavement was originally designed to preclude failure over a reasonable design life, one can compare the two examples. By changing the 18 kip axle load from duals to single tire as illustrated, the pavement life will be shortened when considering both fatigue and rutting. From this analysis, one would expect fatigue to be more critical.

Example 3

Using the results from Example 1, what difference will it make for summer and winter conditions?

Solution

From Example 1, the equivalency was equal to 1.38 and this can be considered the "average" condition in Figure 4. The actual equivalency between 10-inch dual and 18.5-inch single would remain about the same, but actual life to failure (for a particular case) would need to be adjusted. From Figure 4, summer, average, and winter applications for 3-inch asphalt concrete would be 7.4×10^3, 3.3×10^4 and 2.7×10^5, respectively. Therefore, pavement life could be expected to be increased by:

$$\frac{2.7 \times 10^5}{3.3 \times 10^4} = 8.2 \text{ times in winter,}$$

and decreased by

$$\frac{3.3 \times 10^4}{7.4 \times 10^3} = 4.5 \text{ times in summer.}$$

One must realize, however, that the actual range of winter to summer temperatures are distributed throughout the year, month by month, and a weighted average would be more realistic. The data in Figure 4 are primarily for illustrative purposes and show that due to higher stiffness of asphalt concrete in winter, it is more resistant to fatigue cracking.

Example 4

Again using Example 1 results, what is the effect of reducing average truck speed from 70 mph to 55 mph?

Solution

The curve in Figure 5, although developed for the "standard" load, can be used for general speed comparisons. At 70 mph, load applications to fatigue cracking is 1.25×10^6. At 55 mph, this value is 1.15×10^6. Therefore, pavement life is reduced by a factor of

$$\frac{1.15 \times 10^6}{1.25 \times 10^6} = 0.92 \text{ or } 8\%.$$

Example 5

A special highway user requested permission to use a 3-mile segment of highway for trucks having average single axle loads of 24-kips and 18.5-inch single flotation tires. The existing pavement is 3 inches of asphalt concrete with other conditions similar to those developed in this report. What change in pavement would be required to provide equivalent pavement life compared to the standard 10-inch dual, 18-kip axle load?

Solution

Provide an asphalt concrete overlay. Locate the given conditions on Figure 2 (24-kip axle load, 18.5-inch single tire, 3-inch AC pavement). This point is approximately 82×10^{-3} on the vertical equivalency scale. Next, locate the "standard" condition (18-kip, 10-inch dual, 3-inch AC) which is approximately 250×10^{-3}. That is, pavement thickness must be increased sufficiently to increase the 82×10^{-3} to 250×10^{-3}.

By examining the family of AC thickness curves, i.e., 3-in., 6-in. and 9.5-in., it is possible to interpolate any point in between. Therefore, by moving vertically along the 24-kip axle load line from 82×10^{-3} (single 18.5-in. tire) to 250×10^{-3} (dual 10-in. tires), the AC thickness can be interpolated as approximately midway between the 3-in. and 9.5-in. curves, i.e., use 6.2-in. This indicates that 6.2-in. is required and 3-in. is existing (assuming it is new), therefore, a 3.2-in. overlay of this 3-mile section would be required to make it equivalent to the adjoining highway which will receive "standard" traffic.

An additional solution(s) may be more appropriate depending on the conditions:

(1) Add axles to reduce average axle weight.

(2) Change tires to duals (Figure 1 does not include large enough duals--11- or 12-inch would be required by extrapolation, but may not be practicable).

(3) A combination of the above.

Also note that, for larger special loading, the speed may be reduced considerably and this should be considered in the equivalency evaluation. Further, the time of year may be a factor--special hauling may be seasonal and compensation for temperature correction may increase or decrease the equivalency.

SUMMARY AND CONCLUSIONS

This study was initiated in an attempt to examine relative destructive effects of wide single-tire and conventional dual-tires on the pavements. Basic variables are wheel load, tire width and thickness of asphalt concrete. Based on available laboratory and field data three pavement layers consisting of asphalt concrete surface, untreated aggregate base and clay subgrade were selected as the materials for this study. The CHEV 5L program was used to compute deflections and critical strains. Prior to determining the fatigue and rutting equivalencies, maximum computed deflections and critical strains were compared with other sources. These experimental data seem to agree reasonably well. By using known fatigue and failure design curves, maximum allowable numbers of various axle load applications were determined. Fatigue and rutting equivalency relationships for various axle loads are established by dividing the maximum number of axle load applications by the maximum applications of 18-kip axle loads on an asphalt concrete thickness of 6-in. and the dual 5-in. tires. These equivalencies, shown in Figures 2 and 3, can be used to compare the relative pavement life when subjected to various sizes of single and dual tires and axle loads.

The effects of variation in temperature between summer and winter were also examined. It can be said that as the temperature is decreased, as in winter, the rigidity of the pavement structure is increased, thus resulting in a decrease of vertical stress and thereby permitting a greater number of load applications. As can be seen in Figure 4, the destructive effect of a vehicle on the pavement during the summer period is much greater than in winter, exclusive of spring frost break-up conditions and other soil-moisture variations.

The effect of variation in vehicle speed on the pavements was also briefly considered. In general, slower speed tends to cause longer load duration, resulting in an increase in vertical stress and thereby permitting fewer load applications to failure as can be seen in Figure 5. This figure can be used to compare the relative pavement life at various speeds as illustrated in Example 4.

The general nature of Figures 2 through 5 provides a wide range of conditions for comparison on a relative basis. Within reason, interpolation is valid. One must keep firmly in mind, however, the fact that these relationships are for assumed conditions (although reasonable) and do not represent actual pavements. Comparison must be made on a relative basis and not actual pavement life as shown in load applications to failure.

On the basis of this study, several conclusions appear warranted:

1. Single wide flotation tires are in general more destructive than dual tires with equivalent contact area.

2. Similarly, wide flotation tires require a thicker asphalt pavement than dual tires.
3. Pavement requirements for both wide single and dual tires increase at about the same rate as total axle load increases.
4. Pavement life in terms of fatigue is at least an order of magnitude greater in winter than in summer for the conditions of this study.
5. Pavement life is increased directly with speed of the vehicle, all other factors being equal.

ACKNOWLEDGEMENT

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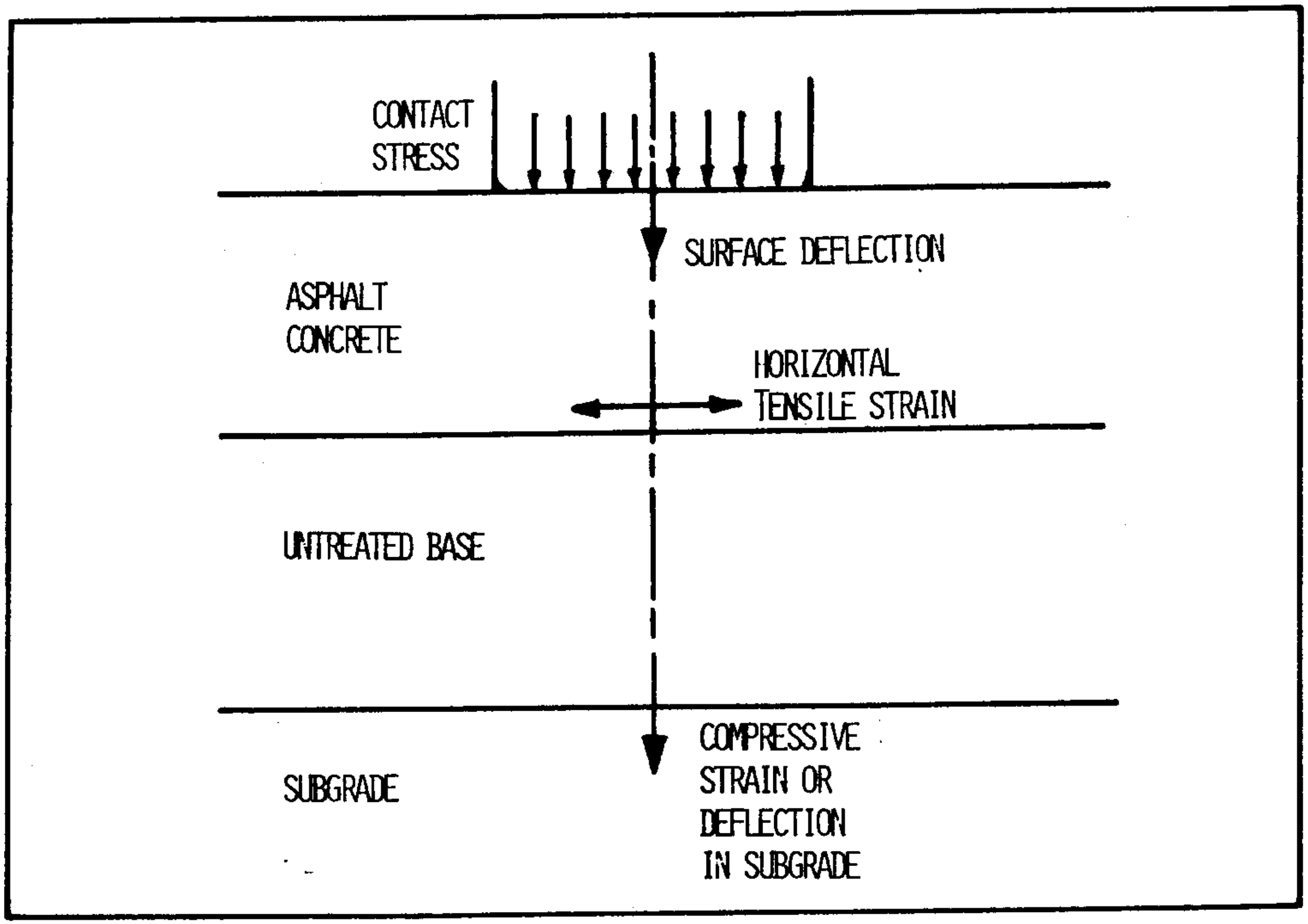

FIGURE 1 - LOCATION OF CRITICAL STRAINS IN PAVEMENT

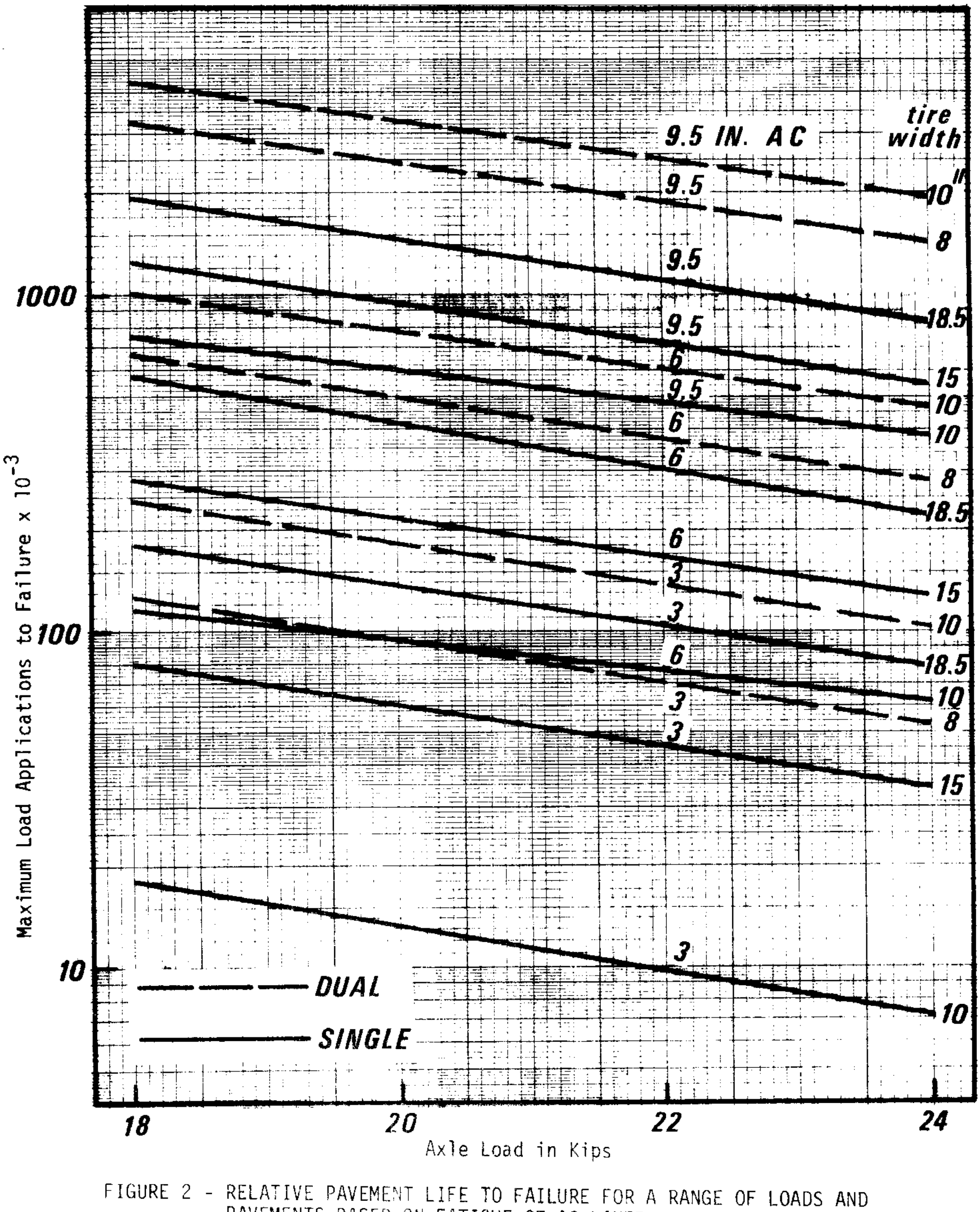

FIGURE 2 - RELATIVE PAVEMENT LIFE TO FAILURE FOR A RANGE OF LOADS AND PAVEMENTS BASED ON FATIGUE OF AC LAYER

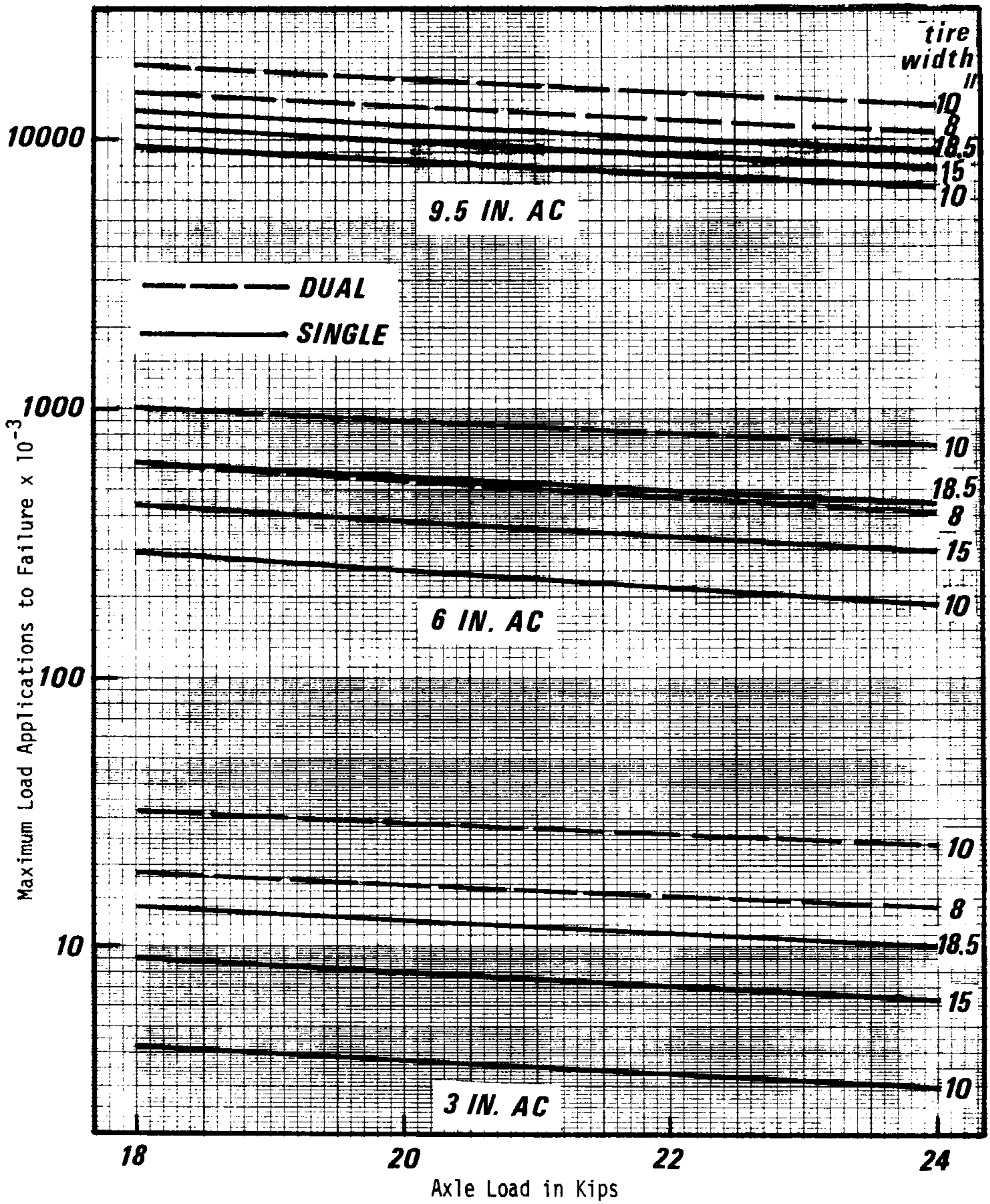

FIGURE 3 - RELATIVE PAVEMENT LIFE TO FAILURE FOR A RANGE OF LOADS AND PAVEMENTS BASED ON RUTTING OF SUBGRADE

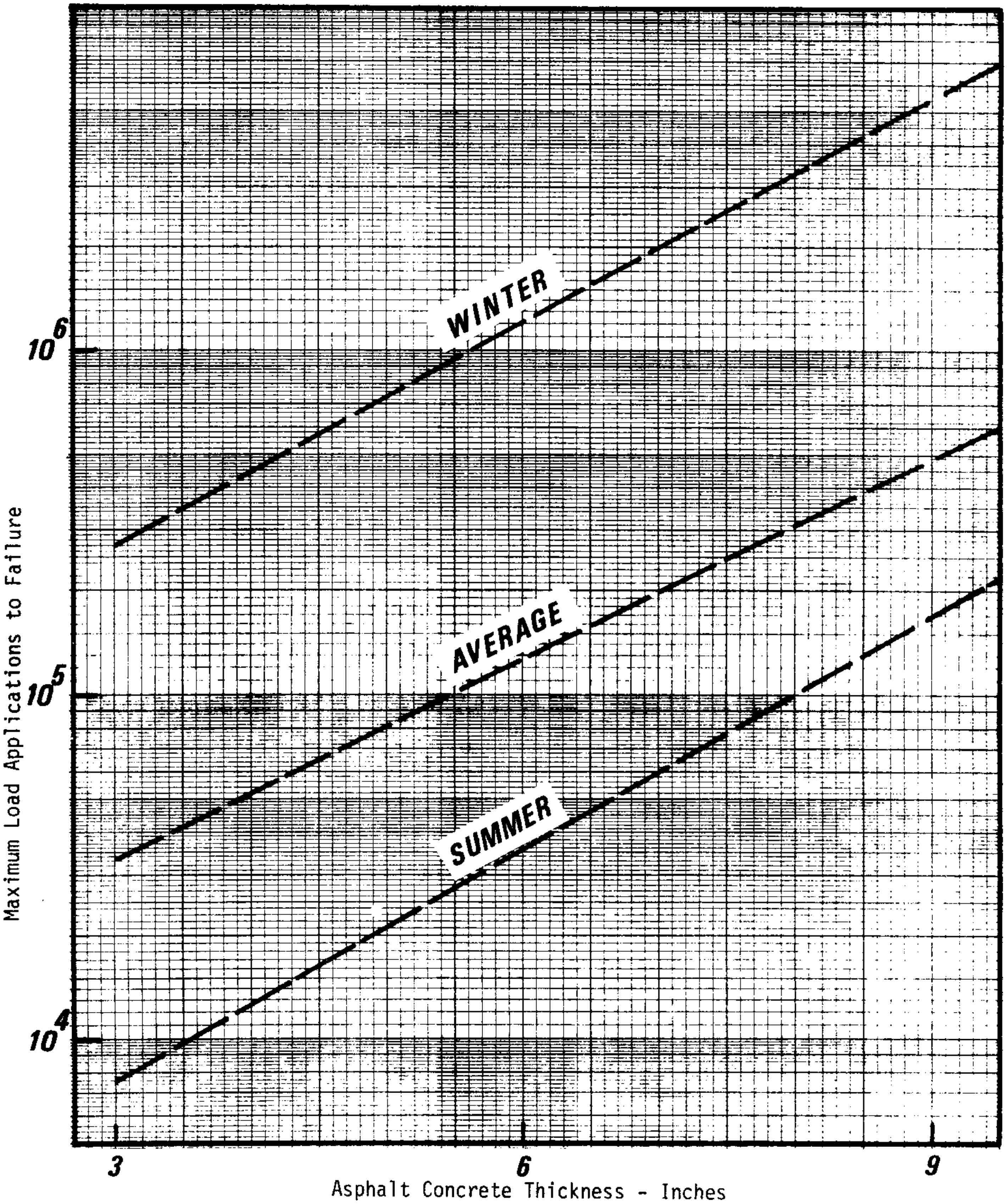

FIGURE 4 - RELATIVE PAVEMENT LIFE OF MAXIMUM ASPHALT CONCRETE FOR SUMMER, AVERAGE AND WINTER

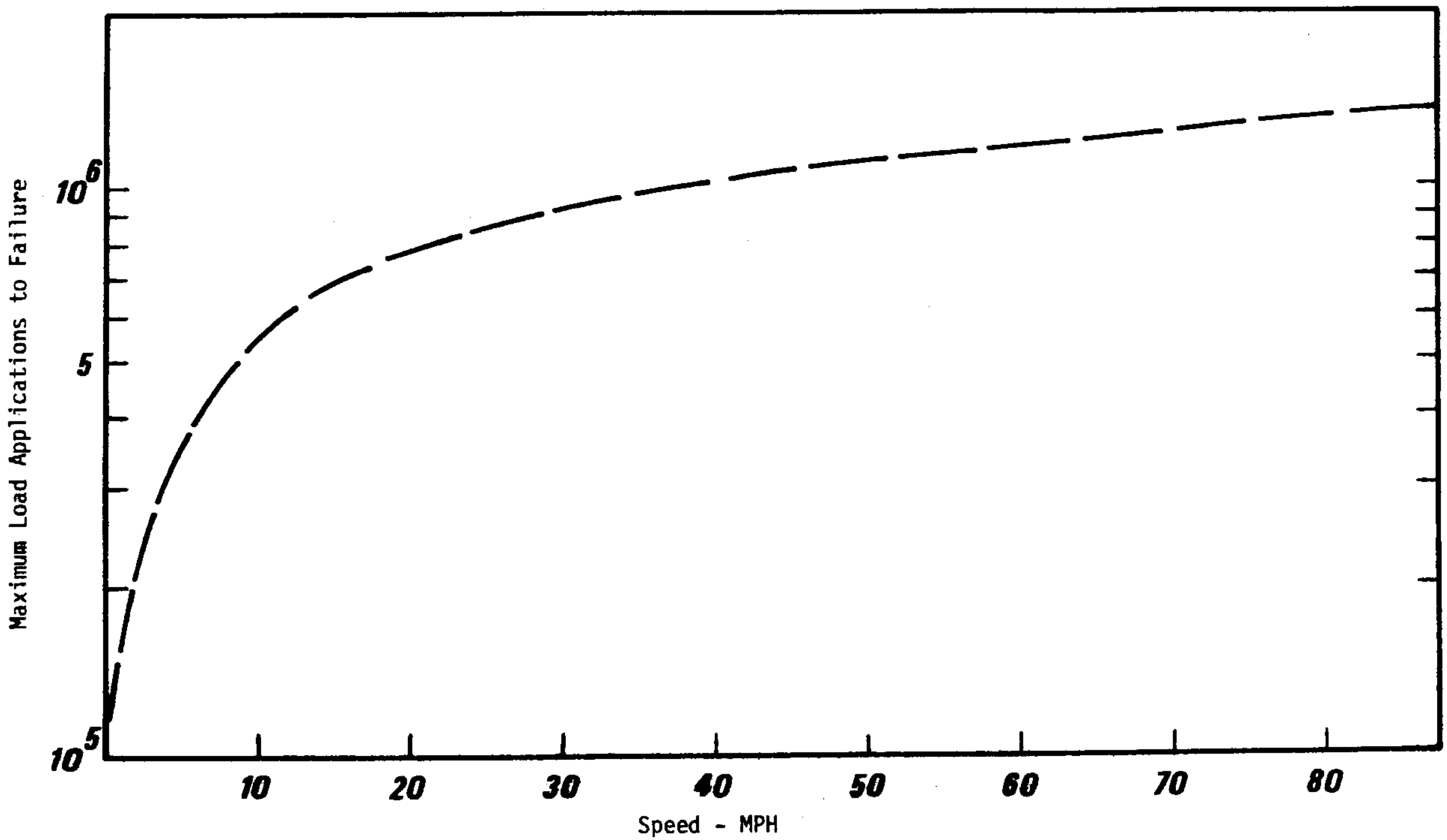

FIGURE 5 - MAXIMUM RELATIVE PAVEMENT LIFE OF ASPHALT CONCRETE FOR RANGE OF VEHICLE SPEEDS