FAA-RD-74-199

DEVELOPMENT OF A STRUCTURAL DESIGN PROCEDURE FOR FLEXIBLE AIRPORT PAVEMENTS

Walter R. Barker and William N. Brabston

U. S. Army Engineer Waterways Experiment Station
Soils and Pavements Laboratory
P. O. Box 631, Vicksburg, Miss. 39180

SEPTEMBER 1975
FINAL REPORT

Document is available to the public through the National Technical Information Service, Springfield, Va. 22151

Prepared for

DEPARTMENT OF DEFENSE
DEPARTMENT OF THE ARMY
Office, Chief of Engineers
Washington D. C. 20314

U. S. DEPARTMENT OF TRANSPORTATION
FEDERAL AVIATION ADMINISTRATION
Systems Research & Development Service

NOTICES

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FAA-RD-74-199	(9) Final rept.,	

4. Title and Subtitle
DEVELOPMENT OF A STRUCTURAL DESIGN PROCEDURE FOR FLEXIBLE AIRPORT PAVEMENTS.

5. Report Date
Sept[redacted]975

6. Performing Organization Code

7. Author(s)
Walter R. Barker, William N. Brabston

8. Performing Organization Report No.
(12) 264 p.

9. Performing Organization Name and Address
U. S. Army Engineer Waterways Experiment Station
Soils and Pavements Laboratory
P. O. Box 631, Vicksburg, Miss. 39180

10. Work Unit No. (TRAIS)
DOT FA73WAI-377

11. Contract or Grant No.

12. Sponsoring Agency Name and Address
U. S. Department of Transportation
Federal Aviation Administration
Department of Defense
Office, Chief of Engineers, U. S. Army
Washington, D.C.

13. Type of Report and Period Covered
Final Report

14. Sponsoring Agency Code
ARD-430

15. Supplementary Notes
(16) RDT/E-4-A-762719-AT-04,
RDT/E-4-A-161102-BJ-2-E

16. Abstract

A design procedure is presented for three types of flexible pavement: conventional, bituminous concrete, and chemically stabilized. These represent nearly all flexible pavements being constructed at this time. Designs are based on analytically determined strain values and experimental and laboratory determined material fatigue strengths. Thus, the procedure can handle in a rational manner the possible variations in the properties of different pavement materials. An adaptation of the cumulative damage concept permits the consideration of cyclic variation in bituminous materials due to variations in temperatures and the variation in subgrade strength resulting from freeze-thaw cycles.

17. Key Words
Pavement design
Flexible pavements
Airport pavements
Paving materials

18. Distribution Statement
Document is available to the public through the National Technical Information Service, Springfield, Va. 22151

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	261	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

038 100

PREFACE

The study described in this report was sponsored by the Federal Aviation Administration as a part of Inter-Agency Agreement No. DOT FA73WAI-377, "New Pavement Design Methodology," and by the Office, Chief of Engineers, U. S. Army, as a part of Military Construction RDTE Project No. 4A762719AT04, "Pavements, Soils, and Foundations," and Military Engineering RDTE Project No. 4A161102BJ2E, "Research in Military Engineering and Construction."

The study was conducted under the general supervision of Mr. James P. Sale, Chief of the Soils and Pavements Laboratory, U. S. Army Engineer Waterways Experiment Station (WES). This report was prepared by Dr. Walter R. Barker and Mr. William N. Brabston.

Director of WES during the study was COL G. H. Hilt, CE. Technical Director was Mr. F. R. Brown.

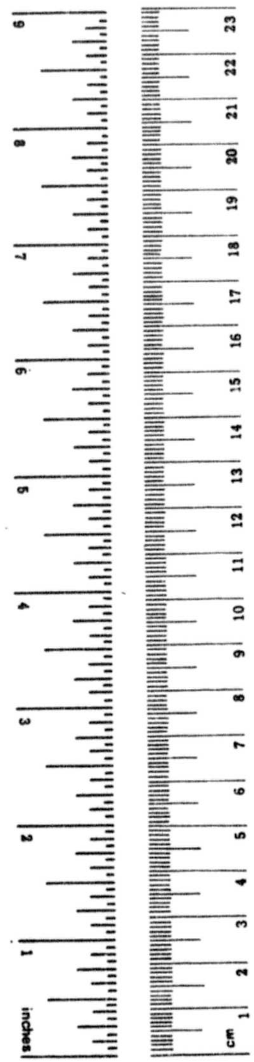
TABLE OF CONTENTS

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

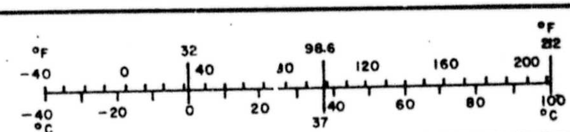
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



INTRODUCTION

BACKGROUND

Design procedures for conventional flexible and rigid airfield pavements and various overlay combinations developed from information obtained during over 30 years of accelerated traffic testing have served civil and military requirements quite well over the years. These procedures were updated and validated during the 1940's and 1950's as gross aircraft weights increased from 75 thousand pounds* to the 1/2- to 3/4-million-pound range. Throughout this period, the procedures provided simple, practical, and useful methods of design for use in all parts of the world and for a wide range of traffic and environmental conditions. During the 1960's, however, the engineering profession began to consider pavement design concepts that departed considerably from the conventional flexible and rigid designs that had been so widely used for civil airports and military airfields.

Applications of these new concepts involved construction with such materials as deep-strength asphaltic concrete; base course and subgrade layers stabilized with cement, asphalt, and fly ash; continuously reinforced concrete; fibrous concrete; and in a few instances, prestressed concrete. As experience was gained with the new approaches, the potentials for cost savings in the life-cycle design of pavements and particularly in a predictive-type design system that would permit consideration of many material usages and maintenance strategies, especially in the interstate highway system, began to become obvious. Even though traffic loadings on highway pavements are considerably less severe than those for military airfields and civil airports, researchers by the late 1960's had begun to think of application of these new concepts to airport and airfield design. A series of research investigations in the late 1960's demonstrated rather conclusively that construction concepts departing radically from conventional flexible and rigid pavement

* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 5.

construction did indeed perform extremely well under aircraft-type loadings up to the range of the C-5A and B-747 aircraft. Unfortunately, it was also obvious from an analysis of these investigations that the empirical design procedures, which had served and could continue to serve satisfactorily for conventional pavement types, were not at all applicable to the stress analyses, deflection analyses or performance predictions for the new construction concepts.

This conclusion emphasized the need to develop and validate a more theoretically based design procedure to accommodate consideration and employment of the new construction concepts, which might be considered to lie somewhere between those for flexible and rigid pavements. The choice of which of the theoretical approaches to use was rather difficult, however, since it was possible to pursue layered linear elastic or viscoelastic analysis, layered nonlinear elastic or viscoelastic analysis, or even absorbed energy theory. In addition, experience gained in the development and use of two- and even three-dimensional finite element analyses increased the confidence that they could also be applied to design problems. All of these approaches had proponents among pavement engineers, but none provided a base that could be considered acceptable throughout the profession. A primary problem was the reluctancy to pursue theoretically based design procedures for a wide range of new pavement types and combinations when the same procedures had not been employed extensively or fully validated against the empirical design procedures for conventional flexible and rigid pavements.

As is obvious from the foregoing remarks, pavement researchers found themselves in a peculiar position. Perhaps an oversimplified explanation of the problem would be as follows: Developing a new design approach for flexible pavements, for example, was probably not worth the effort; however, until a new design approach could be developed to satisfactorily allow for a performance analysis duplicating that for a conventional design, pavement designers would be reluctant to accept the approach for any new system.

Thus, the motiviation for creating, adopting, and employing a theoretically based design procedure for flexible pavement, such as is presented in this report, is related more to the desire to take advantage of new construction concepts than to an attempt to find fault with the empirical procedures presently used in the design of conventional flexible pavements.

PURPOSE AND SCOPE

The purpose of this study was to develop a procedure for the structural design of flexible airport pavements for a given design aircraft. This report presents the procedure and describes its development. The procedure constitutes a new method for determining the required thicknesses of pavement components but does not in any other way change present Federal Aviation Administration (FAA) or Corps of Engineers (CE) design criteria or specifications.

DEFINITIONS

The term "flexible pavement" as used herein refers to a pavement consisting of bituminous concrete placed directly on the subgrade or on either granular or stabilized base and/or subbase course layers. Specifically, the three types of flexible pavement referred to herein are defined as follows:

a. Conventional flexible pavement. A pavement in which all material between the bituminous surface course and the subgrade is unbound granular material and this material serves as the principal structural element for protecting the subgrade.

b. Bituminous concrete pavement. A pavement in which the principal structural element for protecting the subgrade is bituminous concrete or a bituminous-stabilized material. This pavement may or may not have a granular base or subbase course. If the entire pavement structure is composed of high-quality bituminous concrete, then the pavement is referred to as an all-bituminous concrete (ABC) pavement.

c. Chemically stabilized pavement. A pavement in which the base course is composed of stabilized material and the subbase course is composed of either stabilized or unbound granular material. The stabilizing agents generally used in this type pavement are portland cement and lime.

DESIGN PRINCIPLES

The structural deterioration of a flexible pavement is normally associated with cracking of the bituminous surface course and development of ruts in the wheel paths. The design procedure presented in this report handles these two modes of structural deterioration through limiting values of certain response parameters or through accounting for cumulative damage according to Miner's hypothesis. Use of the cumulative damage concept permits handling in a rational manner variations in the bituminous concrete properties and subgrade strength caused by cyclic climatic conditions. As is the case with the limiting values concept, the failure of a pavement system under this concept is assumed to occur when the cumulative damage reaches a fixed amount. This treatment implies that the predicted pavement deterioration is a discontinuous function having the value of "nonfailed" or "failed." Such is not the case for a real pavement, but unfortunately the methodology does not yet exist to predict realistic deterioration functions.

PAVEMENT RESPONSE

In computing the response parameters, the following simplifying assumptions are made:

- a. The pavement is a multilayered structure, and each layer is linear elastic, homogeneous, and isotropic.
- b. The interface between layers is continuous, i.e., the frictional resistance between layers is greater than the developed shear force.
- c. The bottom layer is of infinite thickness.
- d. All loads are circular and uniform over the contact area.

For these assumptions, there are three well-documented layered elastic computer programs which, for a single load, will satisfactorily compute the required pavement response parameters. The three programs are:

- a. BISTRO, developed by the Shell Oil Company.[1]
- b. CHEVRON, developed by the California Research Corporation.[2]

c. CRANLAY, developed by the Commonwealth Scientific and Industrial Research Organization.[3]

Although only the BISTRO program in its original version will compute the response parameters for multiple-wheel loadings, a number of adaptions of the CHEVRON program have been developed for multiple-wheel loadings. One such version developed at the U. S. Army Engineer Waterways Experiment Station (WES) was used in the development of this design procedure.

PAVEMENT RUTTING

In the design procedure, rutting is considered to occur in the subgrade and is controlled by limiting the value of the vertical compressive strain at the top of the subgrade. This assumption implies that the structural layers above the subgrade will be constructed such that only negligible rutting will occur within them. The limiting subgrade strain criteria must be applied to the design of all three types of flexible pavement.

CRACKING OF SURFACE COURSE

The design procedure treats only the load-associated cracking of the bituminous concrete surface course. This type of cracking is treated as being the result of repeated flexural stresses and is controlled by limiting the horizontal tensile strain at the bottom of the surface course. The horizontal tensile strain in this layer is highly dependent on its thickness and on the modulus of the layer immediately beneath it. In order to minimize the required thickness of this layer, the highest possible modulus should be specified for the underlying layer. Present indications are that, for pavements having the minimum surface course thickness specified by FAA and CE criteria and having base course modulus values greater than approximately 70,000 psi, fatigue cracking of the surface course should not be a problem. For all but conventional flexible pavements, it would be rare that the horizontal tensile strain of the surface course would control, as illustrated by the design procedure presented in Izatt, Lettier, and Taylor.[1] The sources of cracking for each of the three types of flexible pavement considered in this report are as follows:

a. In a conventional flexible pavement, cracking is considered to be the result of repeated flexure of the bituminous concrete surface course and is prevented by limiting the horizontal tensile strain at the bottom of this layer.

b. Cracking in a bituminous concrete pavement is assumed to originate at the bottom of the bituminous layer and propagate up to the surface. To control this type of cracking, the horizontal tensile strain at the bottom of this layer is limited.

c. Cracking in a chemically stabilized pavement is generally believed to be the result of reflective cracking from the stabilized material. This type of cracking is minimized by use of a minimum thickness of bituminous concrete surface course or some other special technique to prevent the propagation of cracks from the stabilized material into the surface course. Prevention of this type of cracking is not treated in the proposed design procedure.

TRAFFIC

Operations of a design aircraft are used as the basis for the proposed design procedure. At present, allowance is made for distribution of traffic only in time. The cumulative damage concept allows the consideration of traffic distributed over a time period during which material properties may vary and also provides a methodology by which a more sophisticated treatment can be given for the other dimensions of traffic distribution. Effective handling of the massive data required for considering realistic traffic mixtures and distributions necessitates the use of a computer program for computation and accumulation of the total damage. Development of such a program is now underway, but it has not yet reached a level of confidence satisfactory for incorporation in this design procedure.

MATERIAL CHARACTERIZATION

Pavement materials are characterized in the design procedure by either or both of two parameters, strength and stiffness. The stiffness of the material is characterized by the resilient modulus and Poisson's ratio. The resilient modulus may be determined either in flexure or compression, depending on which method is more appropriate for the particular layer being considered. So far as possible, direct determination of the parameters through laboratory testing should be used in establishing the material characterization. For the cases of granular base and subbase course materials and stabilized material which has cracked and therefore behaves as granular material, the complications introduced through the direct application of laboratory test results require indirect methods for determining modulus values.

DESIGN PROCEDURE

The framework of the design system for a conventional flexible pavement is shown in Figure 1. The input parameters to the system shown are soil data (which may include results of laboratory tests of disturbed and/or undisturbed specimens), traffic data, and climatic data. An iterative process is used to determine an acceptable pavement design. (The frameworks of the design systems for the other pavement types are basically the same. Flow diagrams of the performance models for various flexible pavements are presented later in this report.)

The design system is divided into five major and two minor subsystems. The major subsystems are climate, traffic, material properties, performance, and pavement response. The minor subsystems are initial thickness and thickness modification. For clarity, the subsystem for material properties, as shown in Figure 1, has been broken into bituminous concrete, subgrade, and base and subbase course elements. Each of the subsystems uses specific input and generates output which is required by other subsystems. The subsystems at present are simple, but the entire design system has been developed so that a subsystem can easily be modified or replaced if better methodology becomes available.

CLIMATIC SUBSYSTEM

In the design system two climatic factors, temperature and moisture, are considered to influence the structural behavior of the pavement. Temperature influences the stiffness and fatigue of bituminous materials and is a major factor in frost penetration. Moisture conditions influence the stiffness and strength of the base course, subbase course, and subgrade.

TEMPERATURE EFFECTS ON BITUMINOUS CONCRETE

Temperatures vary widely within the bituminous concrete of the pavement structure. The method of estimating the design pavement temperature for a bituminous concrete layer is shown in Figure 2. Examples of temperature variations in bituminous concrete are presented in

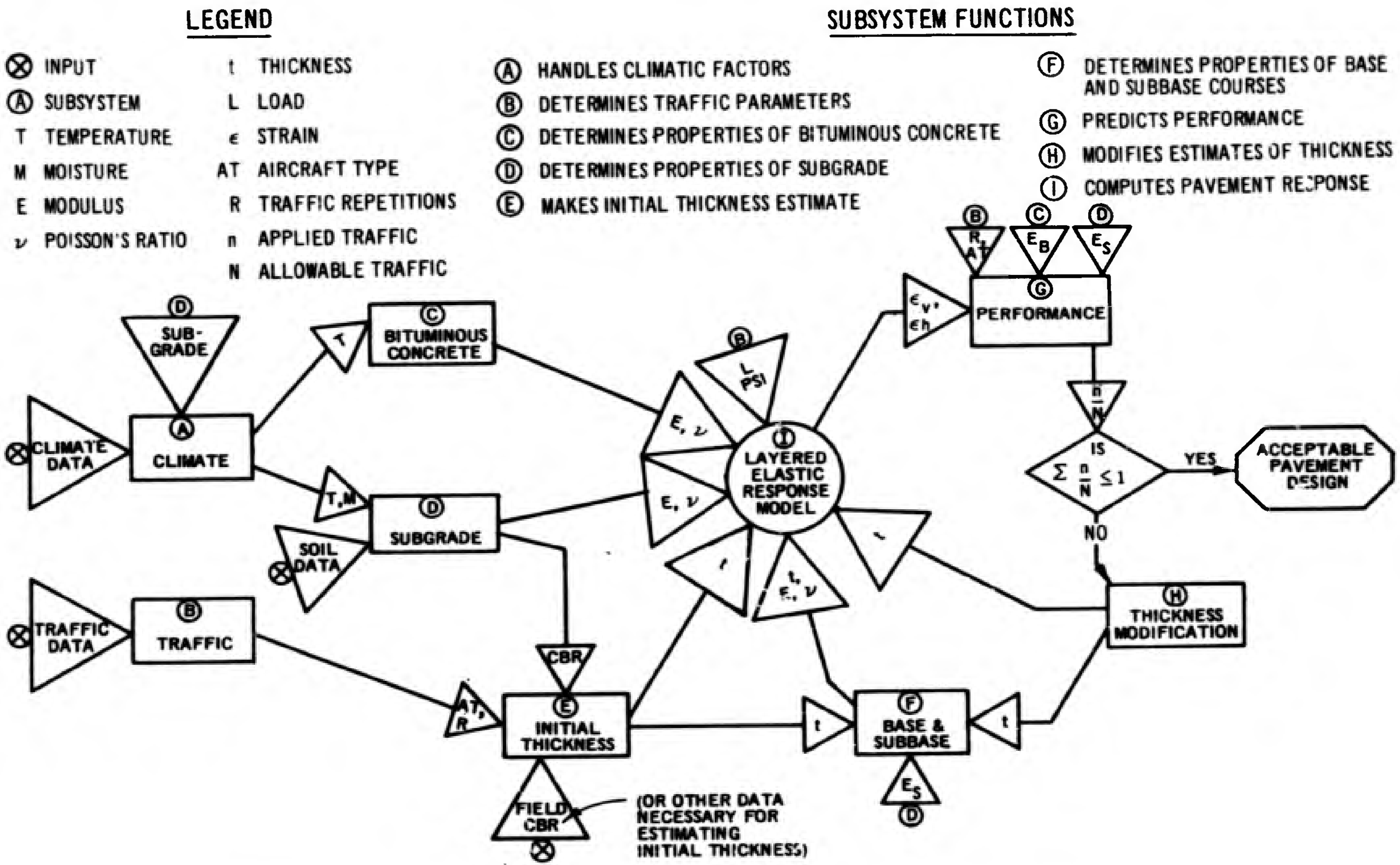


Figure 1. Conventional flexible pavement design system framework

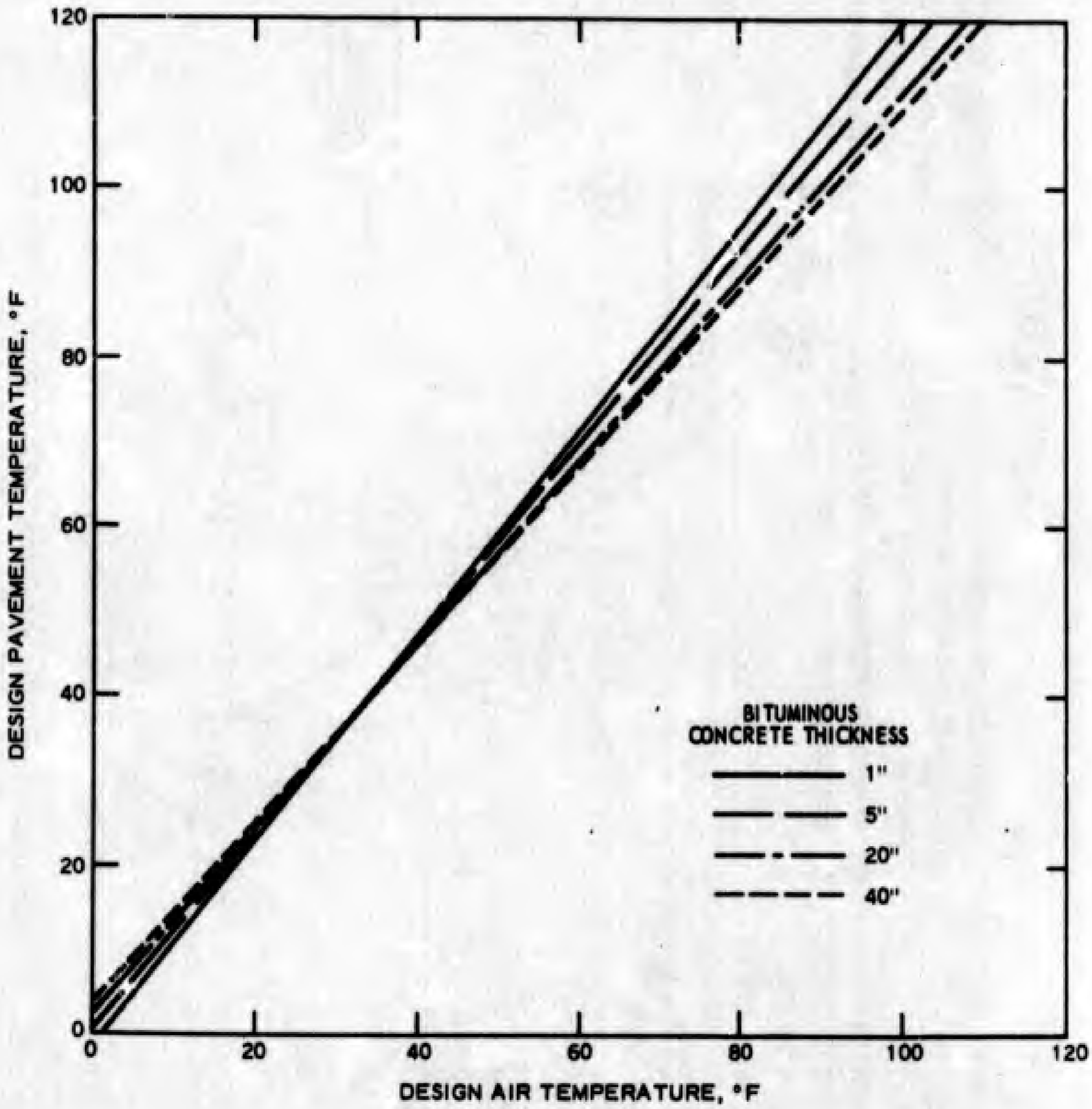


Figure 2. Temperature relationships for selected bituminous concrete thicknesses (after Witczak[4])

Figure 3, which shows the measured temperatures in a 24-in.-thick bituminous concrete pavement located at Vicksburg, Miss.,[5] and in Figures 4 and 5, which show temperatures for a pavement located in Manitoba, Canada.[6]

The design procedure presented in this report requires the determination of one design pavement temperature for consideration of vertical compressive strain at the top of the subgrade and horizontal tensile strain at the bottom of cement- or lime-stabilized layers in a chemically stabilized pavement and a different design pavement temperature for consideration of the fatigue damage of the bituminous concrete surface course. The justification for using different design pavement temperatures is that higher bituminous concrete temperatures are more critical when considering subgrade strain or the fatigue of cement- or lime-stabilized materials, whereas lower pavement temperatures are more critical when considering the fatigue cracking of bituminous materials. In either case, a design air temperature is used initially to determine the design pavement temperature from Figure 2. This method is the same as that used in the design of ABC roads for military facilities as presented in Brabston, Barker, and Harvey.[7]

With respect to subgrade strain and fatigue of cement- and lime-stabilized base or subbase courses, the design air temperature is the average of the average daily mean temperature and the average daily maximum temperature during the traffic period, assuming, of course, that the traffic period is longer than 1 day. For the pavements represented by Figures 3 and 4, the maximum pavement temperatures occurred between 1000 and 2000 hr, which for most airports would correspond to the time period during which the heaviest volume of traffic occurs.

If for the two pavements represented by Figures 3 and 4 the traffic period is 1 day, then the design pavement temperatures can be determined from the data shown. For the pavement located at Vicksburg (Figure 3), the daily mean temperature is 82° F and the daily maximum temperature is 93° F; therefore, the design air temperature would be 87.5° F. From Figure 2, the design pavement temperature would be 99° F. For the pavement located in Manitoba (Figure 4), the average air

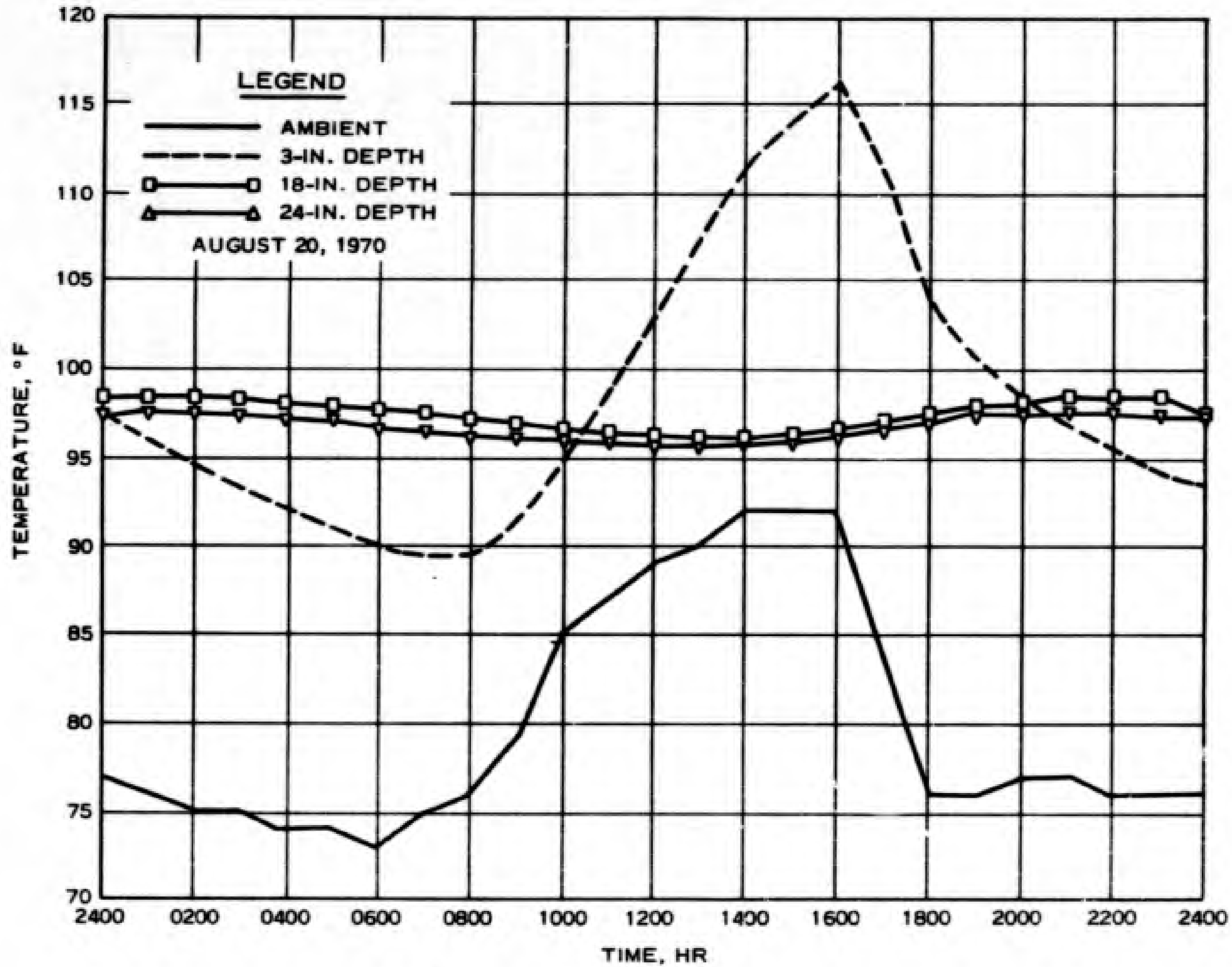


Figure 3. Temperature versus time relationship for a bituminous concrete pavement located at Vicksburg, Miss. (after Burns, Ledbetter, and Grau[5])

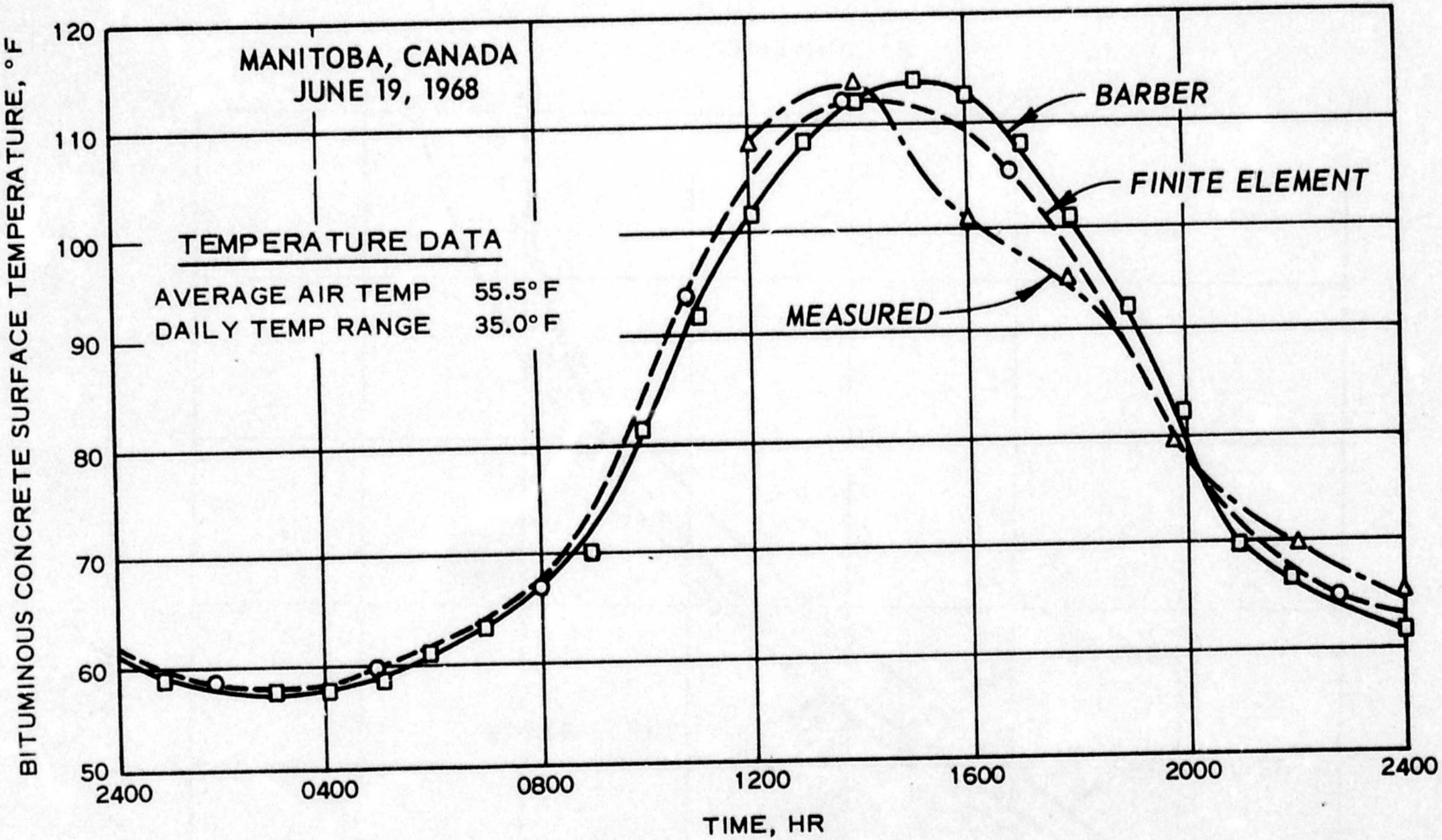


Figure 4. Hourly variation of maximum pavement temperature in a bituminous concrete pavement in Manitoba (after Pretorius[6])

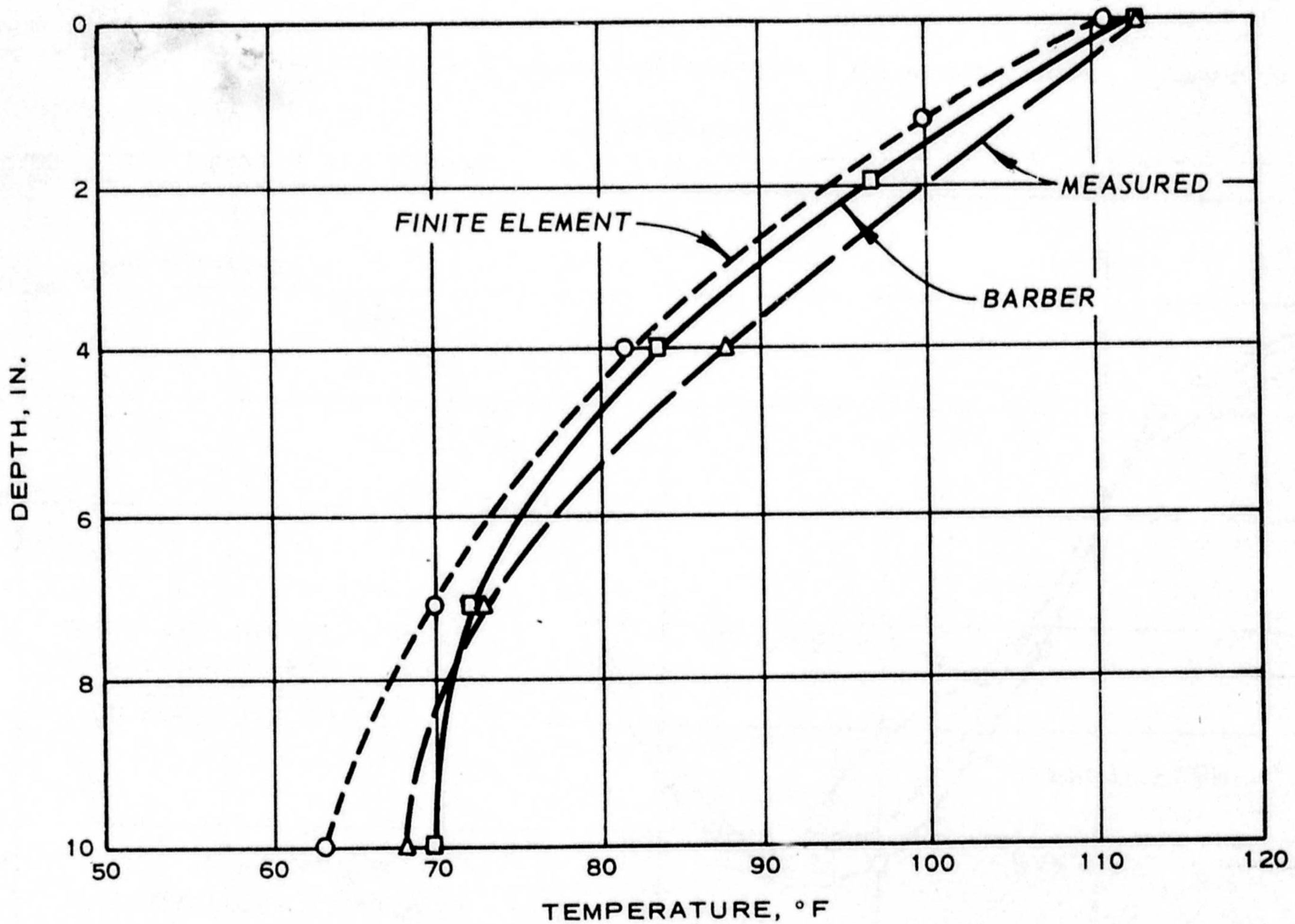


Figure 5. Comparisons of recorded and predicted temperatures (after Pretorius[6])

temperature is 55.5° F and the daily temperature range is 35.0° F. From these data, the maximum daily temperature would be 73° F, and the design air temperature would be 64.3° F. The design pavement temperature would therefore be 75° F. The design pavement temperatures of 99° F for the 24-in. pavement at Vicksburg and 75° F for the 10-in. pavement in Manitoba compare favorably with mean pavement temperatures considering that pavement temperature varies with depth as well as time.

Normally, the traffic period considered in design is expressed in terms of months, and temperature data are therefore obtained from available records. One suggested source of information for such data is the "Local Climatological Data Annual Summary with Comparative Data," which can be obtained from the National Climatic Center, Asheville, N. C. An example of such a summary for Jackson, Miss.,[8] is presented in Appendix A. The design air temperature for the month of August in Vicksburg, Miss., can be determined from the data in Appendix A. For design purposes, it is best to use the long-term averages such as the 30-yr averages given in the annual summary. For the month of August, the average daily mean temperature is 81.5° F and the average daily maximum is 92.5° F; therefore, the design air temperature is 87° F. For a 10-in. bituminous concrete layer, the design pavement temperature for August (determined from Figure 2) would be approximately 100° F. Thus, a design pavement temperature of 100° F would be used to determine the bituminous concrete modulus for the month of August for consideration of subgrade strain or fatigue in cement- or lime-stabilized materials. Examples of the use of the design pavement temperature to determine the modulus of bituminous materials can be found in Brabston, Barker, and Harvey.[7]

For consideration of the fatigue damage of bituminous materials, the design pavement temperature is computed from the average daily mean temperature. The fact that the major portion of traffic would be applied at warm temperatures adds a slight amount of conservatism to the design. Thus, the design air temperature for considering fatigue in the 10-in. bituminous pavement during August for Vicksburg, Miss., would be 81.5° F. The design pavement temperature as determined from Figure 2 would be 92° F.

TEMPERATURE EFFECTS ON SUBGRADE

The effects of temperature on subgrade materials are considered only with regard to frost penetration. The present criteria for determining minimum required thicknesses for frost protection are given in FAA Advisory Circular AC 150/5320-6B[9] and in Department of the Army Technical Manual TM 5-818-2.[10] If frost penetration is a consideration, then the weakened subgrade condition occurring during the thaw period must be taken into account in design. Although the actual thaw period can occur over a relatively short period of time, the weakened subgrade condition is assumed to last for 2 months. For simplicity, the thaw period is assumed to start at the beginning of the month in which the average daily mean temperature is greater than 32° F. Design pavement temperatures for the thaw periods are determined in the same manner as those for the normal period.

A conservative approach is taken with regard to the frozen subgrade condition in that the strength gain due to the subgrade freezing is ignored. For the weakened subgrade condition during the thaw period, the modulus value of the subgrade is determined through appropriate testing of the subgrade materials as described in Appendix B. If the subgrade is permanently frozen, then a maximum allowable subgrade modulus of 30,000 psi may be used.

MOISTURE EFFECTS

In the characterization of base and subbase course materials, no allowance is made for variations in moisture content. The design procedure presented in this report is based on design procedures in which criteria insure adequate drainage of base and subbase courses. Thus, placing granular base material over less porous material, such as a stabilized subbase course, is not permitted. The drainage criteria presently established should be strictly followed.

The resilient modulus of subgrade soils is very sensitive to changes in moisture content. The importance of moisture content in characterizing subgrade soils for design has been pointed out in a report by Monismith and Finn.[11] Also stated in this report is the fact that some gaps exist in design theory, one of which is in the area of prediction of moisture content under pavements. Monismith and Finn recommend that some of the theories be examined more extensively.

Although extensive work is presently under way by different researchers, notably those at the University of Illinois and the University of California at Berkeley, none of the theories have as yet been found applicable to design of airport pavements. CE design in the past has been based on the soaked CBR, which is representative of the worse possible subgrade condition (i.e., a condition in which the subgrade approaches saturation). Until the theories for predicting the moisture content of subgrades have been validated, a conservative approach to characterizing subgrade soils should be taken. In this approach, the subgrade should be considered as saturated and the resilient modulus test should be conducted in accordance with the procedures outlined in Appendix C. If sufficient data are available, resilient modulus tests can be conducted for a range of moisture contents and the modulus value corresponding to the equilibrium moisture content should be used. Sufficient data for such tests would normally consist of field moisture content measurements under similar pavements located in the area. These measurements should be made during the most critical period of the year or when the water table is at its highest elevation. Extreme caution should be exercised when the design is based on other than the saturated condition.

The sensitivity of the modulus to moisture content is illustrated in Figure 6. In this case, for a repeated axial stress of 5 psi, increasing the moisture content from 17.8 to 30.6 percent indicates a decrease in the modulus from 13,000 to approximately 1,200 psi. An increase in moisture content of 23.6 to 27.4 percent causes a 50 percent reduction in the modulus. A reduction in the subgrade modulus affects the design by reducing the allowable subgrade strain and by reducing the modulus of any granular layers above the subgrade, thus resulting in a higher computed subgrade strain. Therefore, it can be seen that the caution recommended with respect to moisture conditions is well justified.

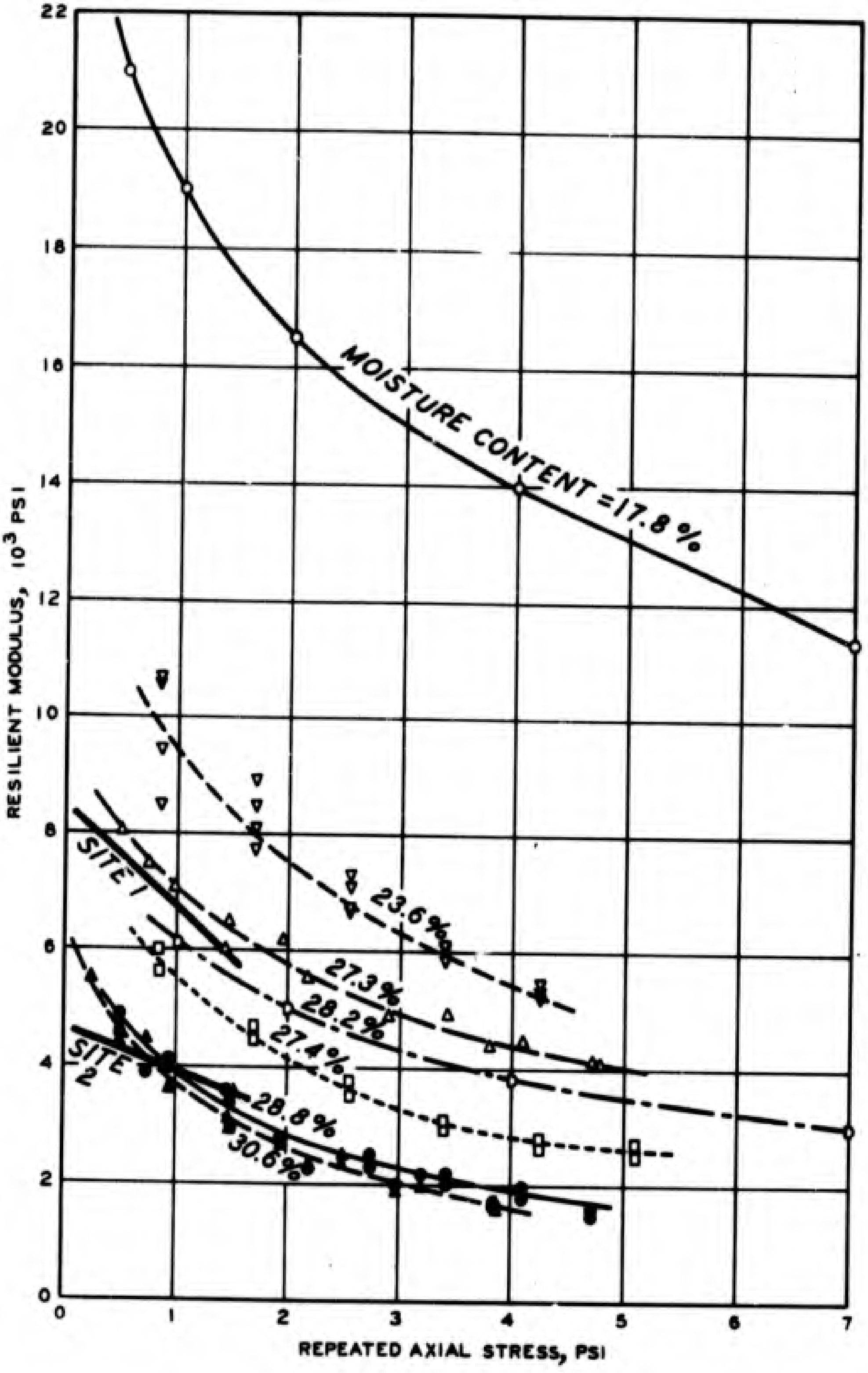


Figure 6. Resilient modulus as a function of repeated axial stress and moisture content (after Fossberg[12])

TRAFFIC SUBSYSTEM

The input parameters for the traffic subsystem are the designation of the design aircraft and the design service life of the pavement in terms of traffic volume. When the design involves mixed aircraft traffic on civil airport pavements, the design traffic volume associated with each type of aircraft is converted to equivalent operations of one aircraft (the design aircraft) following procedures given in AC 150/5320-6B.[9] The design traffic volume for civil airports is expressed in terms of annual "departures" of the design aircraft. The design life of civil airport pavements is normally considered to be 20 yr. For military airfields, the design traffic volume is expressed in terms of total "operations" (passes) expected during the life of the pavement.[13] The term "coverages" is also used in describing the traffic volume on military airfields. The number of coverages is the number of wheel passes applied in the highest density traffic area (normally the center) of the meander width of one main landing gear.

In this design procedure, the traffic volume must be converted to "strain repetitions." For design based on subgrade strain criteria, the annual traffic volume (number of strain repetitions) is used. For design based on horizontal tensile strain in bituminous concrete or stabilized layers, the total traffic volume (again, number of strain repetitions) is used.

The performance criteria (Figure 7) for vertical compressive strain at the top of the subgrade are based on the assumption that, for critical traffic areas (military type A[13] traffic areas), 1 departure or operation (pass) results in the application of 1 strain repetition. In the design of civil airports a thickness reduction factor from AC 150/5320-6B must be applied in the determination of the thickness in noncritical areas. For military airfields in the design of pavements for type B and C traffic areas, the strain criteria as determined from Figure 7 must be increased by a factor of 1.1. For type D traffic areas, the strain as determined from Figure 7 must be increased by a factor of 1.75.

The performance criteria for horizontal tensile strain are based on the total number of strain repetitions, e.g., the repetition level at

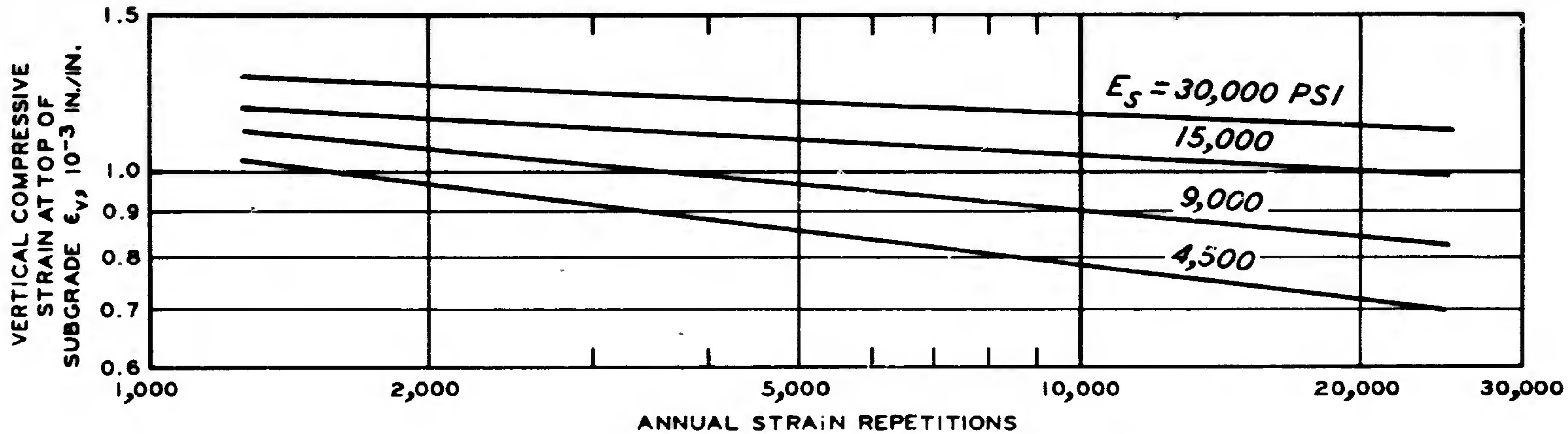


Figure 7. Subgrade strain criteria for design aircraft

failure as determined from a repetitive load flexural beam test. For design of a bituminous concrete surface course or a chemically stabilized base course, in which the location of the critical tensile strain is at a relatively shallow depth in the pavement, total strain repetitions for the design are determined from the number of coverages anticipated over that area of the pavement surface. In this case, 1 coverage constitutes 1 strain repetition. For civil or military purposes, the total number of coverages can be determined from the total number of departures or operations (passes) using the appropriate conversion factor from Table 1. For example, 200,000 departures of a B-747 aircraft would constitute 108,108 coverages, or strain repetitions, using the conversion factor of 1.85. For design of a stabilized subbase course or an ABC pavement, in which the critical tensile strain is located at a greater depth in the pavement, the anticipated traffic volume in terms of departures or operations (passes) may be used directly as the number of strain repetitions. This procedure is applicable to critical traffic areas only and is thus overly conservative for noncritical traffic areas. For design in noncritical areas of civil airports, thickness requirements can be determined by applying a reduction factor from AC 150/5320-6B to the thickness value determined for critical areas and for military airports the strain criteria are increased by the factors given for subgrade strain criteria.

Other data required in the traffic subsystem are obtained based on the type aircraft for which the pavement is designed. The specific data required are the loading, landing gear configuration, and tire contact pressure. Except in traffic areas C and D of military airfields, the aircraft is assumed to be fully loaded. Also the design loading for military aircraft having bicycle gear is increased by 15 percent. Aircraft loads in traffic areas C and D are to be reduced according to TM 5-824-1.[13] Traffic distribution, except as a function of time, is not required, and operations of other aircraft are not considered unless converted to equivalent operations of the design aircraft.

INITIAL THICKNESS SUBSYSTEM

The design system requires an estimated initial thickness to

Table 1

Factors for Converting Departures (Operations) to Coverages For Taxiways and Runway Ends (After Brown and Thompson[14])

Aircraft	Conversion Factor
B-727-00	3.25
B-723-200	3.25
DC-9-30	3.58
B-707-100	1.62
DC-8-10	1.57
DC-10-10	1.82
DC-10-30	1.69
C-880	1.84
B-747	1.85
L-1011	1.81
Single-wheel (FAA)	5.18
A-7D	11.10
C-123	5.23
F-104	11.10
F-4E	8.58
F-111A	4.92
KC-97	3.41
C-124	2.19
C-130	2.09
B-52	1.63
C-135	1.68
C-141	1.72
C-5A	0.81

Note: Coverages = departures (or operations) ÷ conversion factor.

initiate the iterative process. Any available procedure can be used to obtain this estimate. For conventional flexible pavements, present FAA and CE design curves are satisfactory. Estimates of thickness for other types of pavements can easily be obtained through the use of equivalency factors. Equivalency factors for various materials are presented in Table 2. Instructions for use of these factors are given in Hammitt, Barker, and Rone.[15] Use of a specific design procedure to estimate the initial thickness is not absolutely necessary since the design system will work with any reasonable estimate of thickness.

In the thickness determination, it should be remembered that the proposed design procedure does not change the minimum thickness requirements of the present FAA and CE design procedures for bituminous concrete surface course or base course.

In most design situations involving conventional flexible pavements, the presently established minimum thickness for the bituminous concrete surface course is satisfactory. For chemically stabilized pavements having a high modulus, the minimum thickness of the bituminous concrete surface course is established to prevent reflective cracking, which is not treated in this design procedure.

PAVEMENT RESPONSE SUBSYSTEM

To compute the required pavement response parameters (the vertical compressive strain at the top of the subgrade and the horizontal tensile strain at the bottom of the bituminous concrete surface course or stabilized layer), one of the three previously mentioned computer programs can be employed. A modified version of the CHEVRON program was used to develop the subgrade strain criteria, but the BISTRO or CRANLAY programs can also be used. CHEVRON is normally considered to be more economical than BISTRO; however, BISTRO is more accurate than CHEVRON. For conventional flexible pavements in which the modulus ratios are small, the difference in accuracy is negligible. However, when strong stabilized layers having large modulus values are employed in the pavement and the modulus ratios become very large, BISTRO and CHEVRON give significantly different results. The CRANLAY program is a

Table 2

Equivalency Factors for Various Materials (from Hammitt, Barker, and Rone[15])

Material	Equivalency Factor*
ABC	1.70
Bituminous-stabilized GW, GP, GM, GC, SW, SP, SM, and SC	1.50
Cement-stabilized GW, GP, SW, and SP	1.60
Cement-stabilized GM and GC	1.45
Cement-stabilized ML, MH, CL, and CH	1.25
Cement-stabilized SM and SC	1.15
Lime-stabilized ML, MH, CL, and CH	1.10
Lime-and-fly-ash-stabilized ML, MH, CL, and CH	1.15
Unbound crushed stone base course	1.40
Unbound granular subbase course	1.00

* Equivalency factors are based on using optimum percent stabilizing agent for durability and strength.

relatively new program and has not been extensively used; therefore, limited actual use data are available on the program. Other computer programs which meet the basic simplifying assumptions used in developing the criteria can also be used.

MATERIAL PROPERTIES SUBSYSTEM

Material properties required as input to the design system are the modulus of elasticity E, Poisson's ratio ν, and limiting strain ε values. Four general classes of pavement materials are considered: high-quality bituminous concrete, unbound granular base and subbase course materials, chemically stabilized materials in which cementation is the predominant stabilizing mechanism, and subgrade soils. Direct determination of limiting strain values is only applicable for bituminous concrete and stabilized materials.

MODULUS OF ELASTICITY

Bituminous Concrete. Bituminous concrete is a compacted mixture of bitumen and aggregate designed in accordance with Item P-201, "Bituminous Base Course," or Item P-401, "Bituminous Surface Course," of FAA AC 150/5370-10,[16] or CE Guide Specification CE-807.22, "Bituminous Intermediate and Surface Courses for Airfields, Heliports, and Tank Roads (Central-Plant Hot-Mix)."[17] A laboratory test procedure to determine directly the dynamic modulus of similar type mixtures has been used extensively and satisfactorily by several researchers at the Asphalt Institute.[18,19] The dynamic modulus values can be used directly for the modulus of elasticity in a layered elastic pavement model provided appropriate conditions of loading time and temperature are met.

The dynamic modulus is the absolute value of the complex modulus E^* of a linear viscoelastic material such as bituminous concrete.[19] The concept of the complex modulus and a review of the theoretical background associated with it have been presented by Papazian.[20] Briefly, if a linear viscoelastic material is subjected to a sinusoidal loading, the steady state strain response will similarly be sinusoidal in form, at the same frequency, but will lag behind the stress by some phase angle ϕ . The stress-strain relationship for such materials may be expressed in the form of a complex number, when j is the imaginary unit,

$$E^* = E' + jE'' \tag{1}$$

$$|E'| = \frac{\sigma_o}{\varepsilon_o} \cos \phi \tag{2}$$

$$|E''| = j \frac{\sigma_o}{\varepsilon_o} \sin \phi \tag{3}$$

where σ_o is the maximum stress amplitude and ε_o is the maximum recoverable strain amplitude. For complex numbers, by definition,

$$|E^*| = \sqrt{|E'|^2 + |E''|^2} \tag{4}$$

It can be shown that, for high-frequency loadings such as those associated with aircraft ground operations, the phase angle ϕ becomes sufficiently small so that for engineering purposes the expression

$$E^* = \frac{\sigma_o}{\varepsilon_o} \tag{5}$$

is sufficiently accurate to define the modulus of bituminous concrete.[21] This expression implies that for high-speed loadings the material response is essentially elastic in nature.

It is also generally recognized that the stress-strain response of bituminous concrete is a function of the temperature of the material as well as the rate of loading. A well-known general expression for this relationship has been suggested by van der Poel[22] in the form

$$S\ (t,\ T) = \frac{\sigma}{\varepsilon} \tag{6}$$

where

$S\ (t, T)$ = stiffness at a particular temperature T and rate of loading t

σ = axial stress

ε = axial strain

In the test to determine the dynamic modulus, cylindrical specimens of bituminous concrete are subjected to sinusoidal loadings at a rate commensurate with what would be expected in a prototype pavement and over a range of temperatures that would normally be encountered under varying climatic conditions. Specimen preparation techniques, recommended loading and recording equipment, and testing procedures for the dynamic modulus are presented in Appendixes D and E.

In this test, the maximum sinusoidal stress to be applied should be commensurate with the maximum predicted vertical stress expected at the design depth within the bituminous concrete layer. For runway design, it is recommended that a loading rate of 10 Hz be used. For taxiway design, a loading rate of 2 Hz is suggested. These loading rates are appropriate for aircraft speeds of over 100 mph on runways and less than 20 mph on taxiways. It is suggested that specimens be tested at temperatures of 40, 70, and 100° F so that a modulus-temperature relationship can be established. If temperature data as determined by the climatic subsystem described earlier indicate greater extremes than 40 and 100° F, obviously tests should be conducted at these extreme ranges if possible. The modulus value to be used for each strain computation would be the value applicable for the specific pavement temperature determined from the climatic subsystem.

An indirect method for obtaining an estimated modulus value for bituminous concrete is that developed originally by Heukelom and Klomp[23] and later modified by van Draat and Sommer.[24] This method is presented in detail in Appendix F. Use of this method requires that the ring-and-ball softening point and penetration of the bitumen as well as the volume concentration of the aggregate and percent air voids of the compacted mixture be determined. In its original form, this method was considered applicable for mixtures with an air void content up to 3 percent and aggregate volume concentration of 0.7 to 0.9. Van Draat and Sommer suggested use of a corrected aggregate volume concentration value for mixtures with air void concentrations in excess of 3 percent and presented a constraint relationship with respect to volumetric relationships between the bitumen and the aggregate. These concepts are incorporated in the method presented in Appendix F.

Unbound Granular Base and Subbase Course Materials. The terms "unbound granular base course material" and "unbound granular subbase course material" as used herein refer to materials meeting grading requirements and other requirements in applicable CE and FAA specifications. The method of characterizing such materials based on performance analyses of field test pavements having similar base and subbase course layers is described later in this report. In this method, a modulus value is first determined for the subgrade then the modulus value of the next overlying layer or sublayer is determined from a plot, depending on the type of material and thickness of the upper layer. This procedure is repeated to determine the modulus values for the full base and subbase course thicknesses. Use of this method to determine modulus values is presented in detail in Appendix G.

Stabilized Material. The term "stabilized material" as used herein refers to soil treated with such agents as bitumen, portland cement, slaked or hydrated lime, fly ash, etc., or a combination of such agents to obtain a substantial increase in the strength of the material over its untreated natural strength. Chemically treated soils in which no substantial increase in strength is obtained should be characterized using the methods presented herein for unbound base, subbase, and subgrade materials. Chemically treated soils having unconfined compressive strengths greater than 250 psi are considered to be stabilized materials and should be tested in accordance with the methods specified for stabilized materials. Chemically treated soils having unconfined compressive strengths less than 250 psi are considered to be modified subgrade soils and should be tested under the provisions for subgrade soils. Bituminous-stabilized materials should be characterized in the same manner as bituminous concrete. Soils stabilized with other chemicals, particularly those in which pozzolanic action is present, should be characterized using flexural beam tests or cracked section criteria described later in this report. Flexural modulus values determined directly from laboratory tests can be used when the effect of cracking is not significant and the computed strain based on this modulus does not exceed the allowable strain for the material being used. Investigations of the fatigue properties of soils stabilized with portland cement using flexural beam procedures have been described by several investigators, including Wang[25] and Pretorious.[6]

These methods do not appear to be limited to use with cement-treated soils, however, and should be applicable to other types of chemically stabilized materials.

The general approach in the flexural beam test is to subject the specimen to repeated loadings at third points, measure the maximum deflection at the center of the beam (i.e., at the midpoint of the neutral axis), and calculate the values for the flexural modulus based on the theory of a simply supported beam. A correlation factor for stress is applied.

Procedures for preparing specimens of and conducting flexural beam tests on chemically stabilized soils are presented in detail in Appendix H. Criteria for determination of a modulus value based on a cracked section are also presented in Appendix H.

Criteria for cement-stabilized soils to be used as base and subbase course materials in airport pavements are presented in the following tabulation:

Stabilized Layer	Unconfined Compressive Strength, psi For Cited Design Aircraft Loading, kips		
	<30	30 to 200	>200
Base course	500	1000	1000
Subbase course	250	500	500

For military airfields, the design weight of the controlling aircraft category must be known in order to enter the applicable column in this tabulation. These criteria indicate the minimum unconfined compressive strength required of laboratory prepared specimens in order for the treated material to be considered fully stabilized and of adequate structural quality. The minimum specified strength must be obtained on specimens tested after a 7-day moist cure in accordance with American Society for Testing and Materials Designation: D 1633.[26] In addition to strength requirements, stabilized base and subbase course materials must also meet other provisions in applicable CE and FAA specifications.

Subgrade Soils. The term "subgrade" as used herein refers to the natural, processed, or fill soil foundation on which a pavement structure is placed. A suitable method for characterizing such material in the laboratory is the resilient modulus test using procedures developed by Seed and Fead.[27] A complete description of the tests and a study of the effect of various soil parameters and properties on test results is presented in Seed, Chan, and Lee.[28] The basic laboratory technique described earlier for conducting dynamic modulus tests on bituminous concrete is essentially an offspring of the original procedure developed by Seed and Fead[27] for soils; therefore, the general principles of both procedures are similar.

For subgrade soils, the modulus is not dependent on temperature; however, subgrade materials are affected by changes in moisture content. Therefore, the test procedures presented herein contain provisions for saturation of soils that are moisture-susceptible. In normal airport construction, the subgrade soil is compacted to 95 to 100 percent of modified American Association of State Highway and Transportation Officials (AASHTO) maximum density and at or near the optimum moisture content for that compaction effort. As a result of normal moisture migration, water table fluctuation, etc., the moisture content of the subgrade soil can increase and approach saturation with only a slight change in density. Since the strength and stiffness of fine-grained materials are particularly affected by such an increase in moisture content, it is desirable to test these soils in the near saturation state. Two methods are available to obtain a specimen with this moisture content: the soil can be either molded at optimum moisture content and subsequently saturated, or molded at the higher moisture content using static compaction methods. Although there is some evidence that the resilient properties of both types of specimens are similar, most of the tests reported in the literature involve materials compacted using the standard AASHTO compaction effort. It is not apparent whether this concept is valid for materials compacted at the higher densities; therefore, for the test procedures presented herein, the method recommended for developing high moisture contents in test specimens is back-pressure saturation. Procedures for specimen preparation, testing, and interpretation of test results are presented in Appendix C. For this design procedure, however, the maximum allowable modulus for a subgrade soil should be restricted to 30,000 psi.

POISSON'S RATIO

Due to the complexity of laboratory procedures involved in the direct determination of Poisson's ratio for pavement materials and the relative influence on pavement design of this parameter when compared with other parameters, use of values commonly recognized as being acceptable is recommended. These values for the four classes of pavement materials considered herein are presented in the following tabulation:

Pavement Material	Poisson's Ratio ν
Bituminous concrete	0.5 for E <500,000 psi 0.3 for E >500,000 psi
Unbound granular base or subbase course	0.3
Chemically stabilized base or subbase course	0.2
Cohesive subgrade	0.4
Cohesionless subgrade	0.3

LIMITING STRAIN CRITERIA

Bituminous Concrete. The primary means recommended for determining values of limiting horizontal tensile strain for bituminous concrete is the use of the repeated load flexural beam test on laboratory prepared specimens. Procedures for this test are presented in detail in Appendix I. The basic test procedures are similar to those described by Deacon and Monismith[29] except that, instead of a pneumatic pressure system, an electrohydraulic system similar to that used by Kallas and Riley[18] is specified. Basically, the test involves applying a repetitive two-point loading to a laboratory prepared beam specimen under controlled stress and constant temperature conditions until failure occurs. A number of tests are run at different stress levels, and the data are presented in the form of a plot of initial bending stress versus load repetitions to fracture. For each temperature condition, therefore, a value of limiting horizontal tensile strain can be determined for the pavement design life.

An alternative method of determining limiting horizontal tensile strain for bituminous concrete is the use of the provisional laboratory fatigue data employed by Heukelom and Klomp.[30] These data are presented in Appendix I in the form of relationships between stress, strain, load repetitions, and elastic moduli of bituminous concrete.

Stabilized Soils. Limiting horizontal tensile strain criteria and flexural modulus values for stabilized soils may be determined from laboratory tests on flexural beam specimens as specified in Appendix H. If the strain value determined from the laboratory repetitive load test is less than 1.5 times the computed strain (where the strain has been computed by the response model using the flexural modulus), then the procedure described in Appendix H should be followed to obtain an equivalent cracked section modulus value. The procedure for comparing computed strains to allowable strain is discussed in more detail in the following paragraphs.

PERFORMANCE SUBSYSTEM

In principle, the performance prediction should be based on the cumulative damage for the number of design aircraft operations; however, the massive amount of data required for such an approach makes it unrealistic. To make the system workable, traffic repetitions are grouped into periods during which it is assumed that each aircraft operation will cause an equal amount of damage.

The design procedure uses cumulative damage to account for variation of two design parameters. These two parameters are the decrease in subgrade strength due to thawing from a frozen state and the variation in the properties of bituminous concrete due to changes in the temperature. Cumulative damage and failure are predicted by the linear summation of the ratios of the applied traffic repetitions to the allowable traffic repetitions. Failure is predicted at the traffic level at which the summation of the ratios equals one. That is, the criteria for design are such that

$$\sum_{i=1}^{k} \frac{n_i}{N_i} = 1 \qquad (7)$$

where

k = number of periods into which the traffic is grouped

n_i = applied traffic repetitions

N_i = allowable traffic repetitions

With respect to subgrade strain, traffic is defined as the number of annual departures or operations of the design aircraft to be applied over a 20-yr life. Thus, if the design is for 6000 annual departures and the traffic is distributed evenly throughout the year, n for a 1-month period would be $6000 \div 12 = 500$. Based on the computed subgrade strain, the value of N would then be selected from Figure 7 in terms of annual strain repetitions.

When the tensile strain in bituminous concrete or stabilized materials is considered, the value of N is the number of strain repetitions at which failure would be expected at the computed strain level. Traffic would then assume the meaning of a measurement of the number of horizontal tensile strain repetitions. For strains at shallow depths in the pavement, such as in bituminous concrete surface course, coverages would be the appropriate measure of traffic, i.e., n would be the total number of coverages for a specific period. For horizontal tensile strains at greater depths, such as at the bottom of stabilized layers, the total number of aircraft departures or operations (passes) during the period would be applicable, i.e., n would be the total number of departures.

For the design system, three types of strain values are to be considered: the horizontal tensile strain at the bottom of the bituminous concrete; the vertical compressive strain at the top of the subgrade; and, for pavements containing stabilized layers, the horizontal tensile strain at the bottom of the stabilized material. Each of these criteria must be checked, and, ideally, for a balanced design, the summation of ratios should reach the value of unity at the same traffic level. In practice, it will be found that one of the criteria will control the design.

In computations of cumulative damage, it should be remembered that the system is attempting to account for the damage due to each aircraft operation and that the pavement properties are different for each operation. The periods during which traffic can be grouped and the validity of grouping traffic depend on the pavement type and the particular design situation.

The selection of traffic periods should be limited to periods during which there is relatively little variation in the values of the critical design parameters. Thus, if cumulative damage is to account for variations in the properties of the bituminous concrete caused by cyclic temperature variations, then the traffic grouping should be limited to the time during which the temperature of the bituminous concrete can be represented by a single value.

Performance models for the types of flexible pavement considered herein are discussed in the following paragraphs, and a flow diagram for each pavement type is presented.

CONVENTIONAL FLEXIBLE PAVEMENTS

For conventional flexible pavements, the horizontal tensile strain at the bottom of the bituminous concrete surface course and the vertical compressive strain at the top of the subgrade are the design strains. In consideration of the subgrade strain, the influence of the variation in the modulus of the bituminous layer is normally ignored. Such treatment is justified for two reasons. First, the computed subgrade strain is insensitive to the modulus of the relatively thin layer of bituminous concrete. Second, the allowable subgrade strain is insensitive to the time distribution of the traffic volume and thus traffic may be grouped into a critical time period. If the subgrade is subjected to freezing and thawing, then the damage for both the normal and the thaw periods must be computed. Thus, for a conventional flexible pavement in which the freeze-thaw cycle is not a consideration, the system is reduced to a comparison of the allowable subgrade strain selected from Figure 8 with the computed strain. When the freeze-thaw cycle is a consideration, the cumulative damage must be determined for both the normal and the thaw periods. No allowance is made for an increase in subgrade strength due to freezing.

For this situation, the value of n is the number of applied

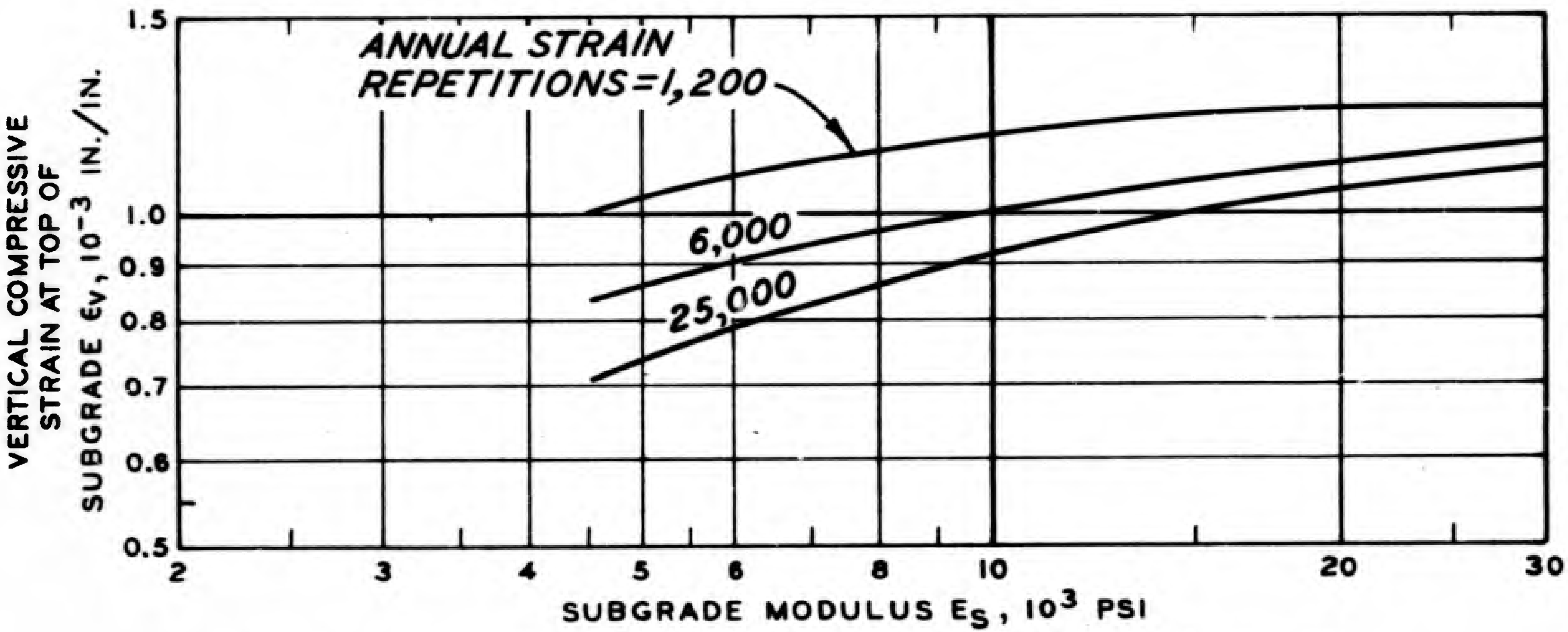


Figure 8. Limiting subgrade strain criteria for conventional flexible pavements

annual strain repetitions during the respective periods. A period of 2 months during the spring is assumed to be the thaw period, during which the subgrade strength is represented by the thaw resilient modulus. The remaining 10-month period is assumed to be the normal period, during which the subgrade strength is represented by the saturated modulus. The subgrade strains are computed for each period, and the allowable annual strain repetitions N for each condition are determined from Figure 8. For a satisfactory design, the sum of n/N for the thaw and normal periods should not exceed one.

In a consideration of the horizontal tensile strain in the bituminous concrete, the variation in the properties of bituminous concrete due to temperature variation becomes critical; therefore, damage must be accumulated over shorter periods of time. For the procedure presented in this report, it is suggested that periods of 1 month be used as grouping periods. The criteria for a 20-year life would thus be such that

$$20\left(\sum_{i=1}^{12} \frac{n_i}{N_i}\right) \leq 1 \qquad (8)$$

where n_i is the total number of applied strain repetitions during the given month i and N_i is the number of allowable strain repetitions determined from the computed strain and bituminous concrete properties for the month i . In the computation of horizontal tensile strain, the bituminous concrete modulus is determined based on the average daily mean temperature for a certain time period. In the case in which thaw is not a problem, the subgrade modulus is a constant. If thaw is a problem, then the subgrade modulus must be reduced for a 2-month period following the thaw.

A flow diagram of the performance model for conventional flexible pavements is shown in Figure 9.

BITUMINOUS CONCRETE PAVEMENTS

In bituminous concrete pavements, the variation in the modulus due to variations in temperature becomes a consideration in the evaluation of subgrade strain as well as the horizontal tensile strain. This

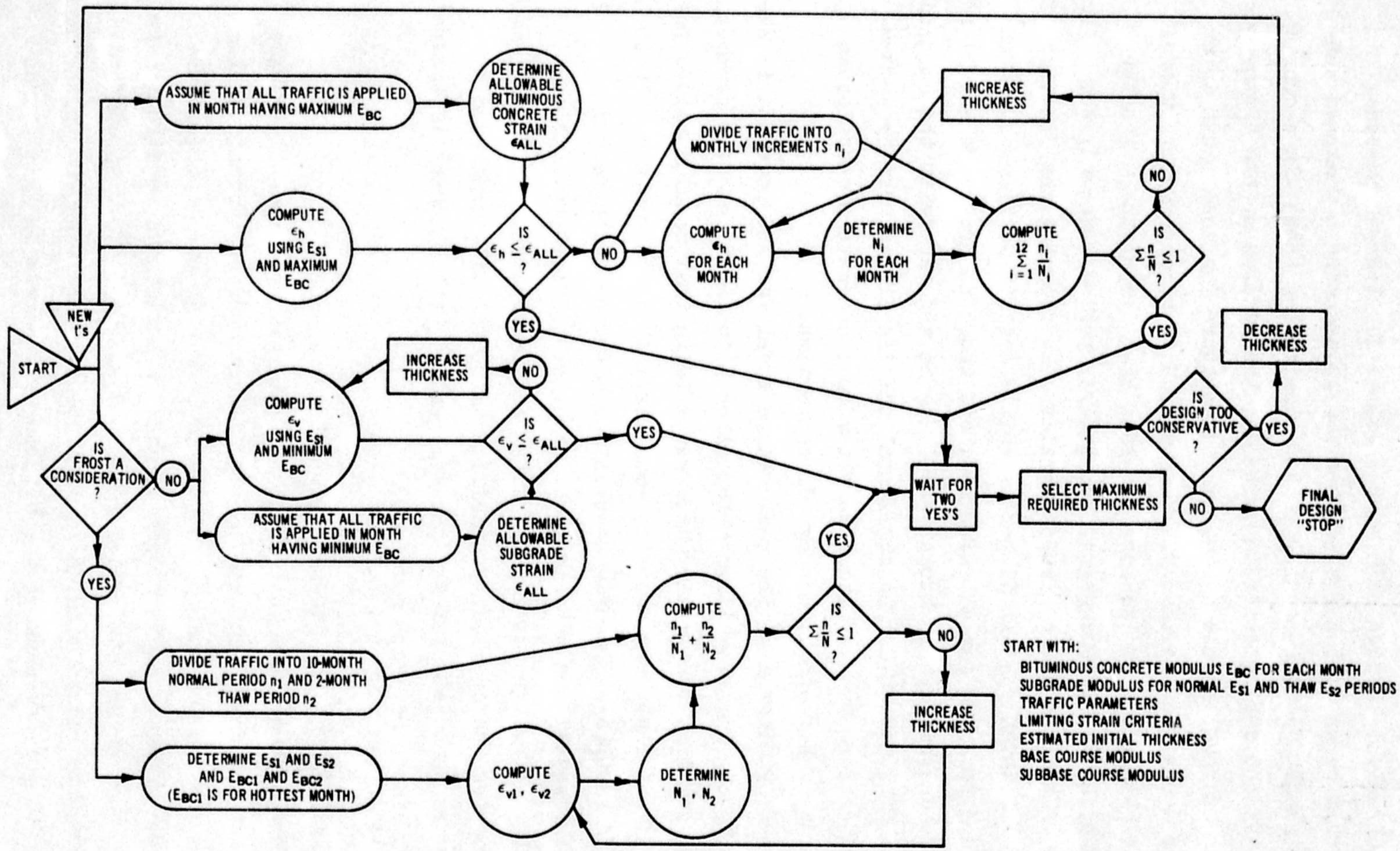


Figure 9. Flow diagram of important events for conventional flexible pavement

is accounted for through summing the damage for 1-month periods. Freeze-thaw conditions are accounted for by assuming a weakened subgrade strength for a 2-month period following the thaw. In a consideration of the subgrade strain criteria, the modulus of the bituminous concrete is based on the design air temperature for each month determined as described previously. This method is a conservative approach that was chosen for two reasons. First, high temperatures greatly influence the damage occurring over a given period. Second, the major portion of traffic at an airport usually occurs during that portion of the day for which the air temperature is above the daily mean. Such may not be the case for all airports, and thus the procedure should be slightly conservative in those cases. As is the case with the subgrade criteria, the damage due to horizontal tensile strain must be accumulated to reflect the variation in bituminous concrete properties and, if thawing is a factor, in subgrade properties.

A flow diagram of the performance model for ABC pavement is shown in Figure 10. The flow diagram for other bituminous concrete pavements would be similar, the primary difference being that modulus values for any base and/or subbase material would have to be determined. If the base and/or subbase directly beneath the bituminous concrete is a chemically stabilized material, the pavement is first treated as a chemically stabilized pavement. After the determination has been made as to whether the chemically stabilized material will crack and a modulus has been selected for these materials, the pavement is treated as a bituminous concrete pavement.

CHEMICALLY STABILIZED PAVEMENTS WITH STABILIZED BASE COURSE AND UNSTABILIZED SUBBASE COURSE

For a chemically stabilized pavement having a stabilized base course and a granular subbase course, damage must be accumulated for subgrade strain, for horizontal tensile strain at the bottom of the bituminous concrete surface course, and for horizontal tensile strain at the bottom of the stabilized layer. Normally in this type of pavement, the base course modulus is sufficiently high (≥100,000 psi) to prevent fatigue cracking of the bituminous concrete surface course

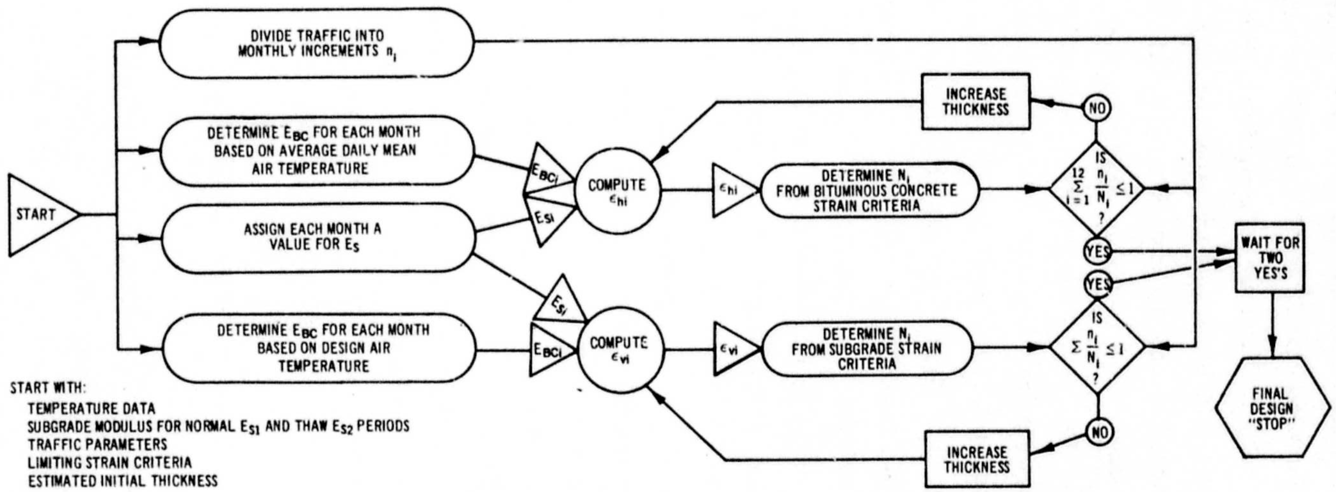


Figure 10. Flow diagram of important events for ABC pavement

(where the bituminous concrete surface course has a thickness equal to or greater than the minimum required by CE and FAA criteria), and thus this mode of failure is only a minor consideration. For most cases, a very conservative approach can be taken in checking for this mode of failure, i.e., all the traffic can be grouped into the most critical time period and the computed bituminous concrete strain compared with the allowable strain. If the conservative approach indicates that the surface course is unsatisfactory, then the damage can be accumulated in the same manner as that used for conventional flexible pavement.

For a pavement of this type, the criteria become more complicated than those of conventional flexible or bituminous concrete pavements. Two cases in particular should be considered. First is that in which the stabilized layer is considered to be continuous, with only shrinkage cracking due to curing and temperature. The second case is that in which the stabilized layer is considered cracked due to load. The first step in evaluating the stabilized layer is to compute the horizontal tensile strain at the bottom of the stabilized layer and the vertical compressive strain at the top of the subgrade, assuming the stabilized layer to be continuous and having a modulus value as determined by the flexural modulus test. To account for the increase in stress due to loadings near shrinkage cracks, the computed strain should be multiplied by 1.5 for comparison with the allowable strain.[31] If the analysis shows that the stabilized base will not crack under load, then it will be necessary to compare the adjusted value of subgrade strain with the allowable subgrade strain. If this analysis indicates that the adjusted strain is not less than or equal to the allowable strain, then the thickness should be increased and the process repeated, or the section should be checked by assuming that the base course will crack and behave as a granular material. The cracked stabilized base course is represented by a reduced modulus value which is determined from the relationship between modulus and unconfined compressive strength shown in Figure 11. The relationship shown in Figure 11 was developed from data from a limited number of field test sections at WES, and the procedure presented should be used with caution. When the cracked base concept

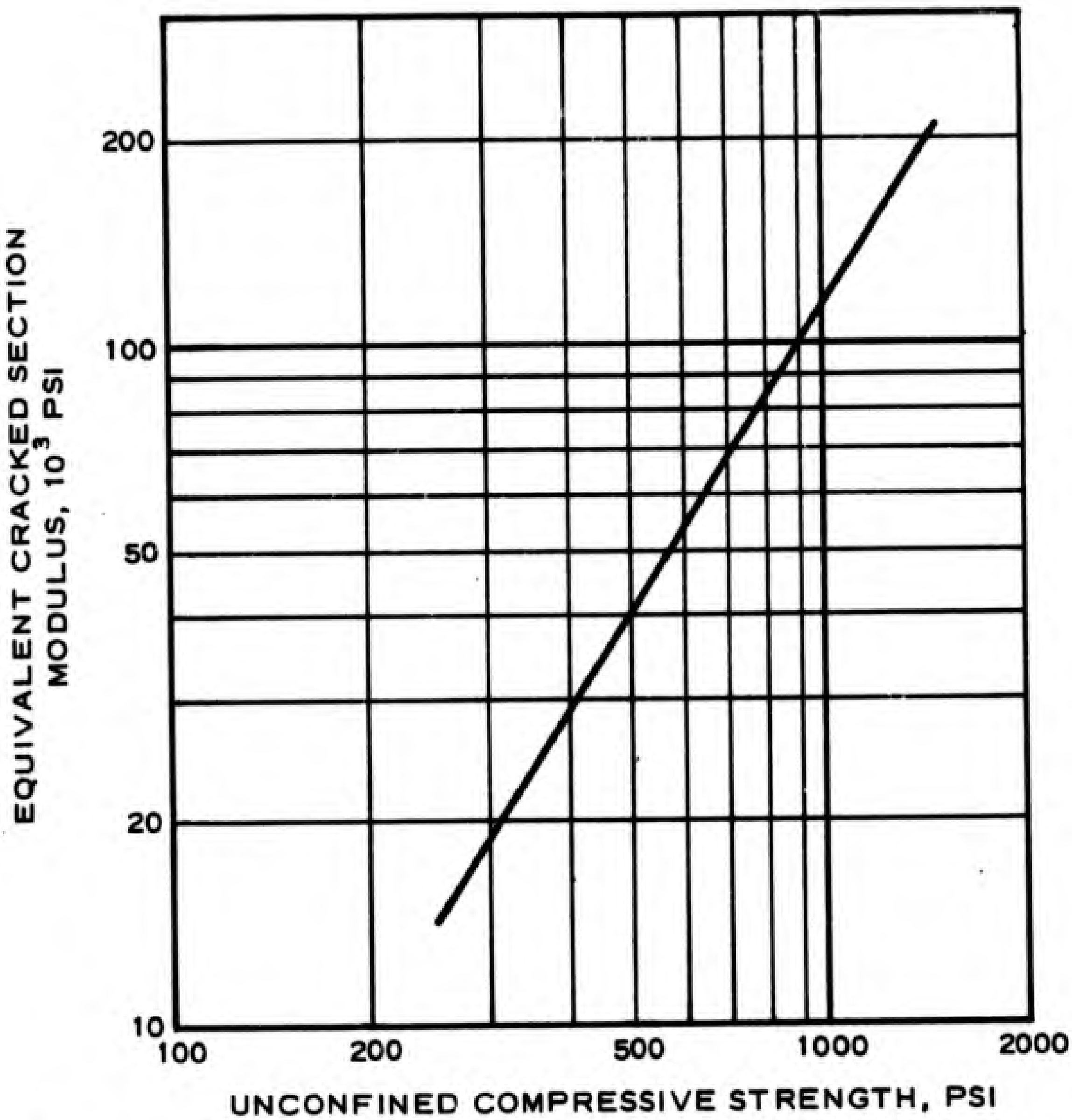


Figure 11. Relationship between equivalent cracked section modulus and unconfined compressive strength

is used, only the subgrade criteria need to be satisfied. The section obtained should not differ greatly from the section obtained by using the equivalency factors (Table 2) developed by Hammitt, Barker, and Rone.[15] A flow diagram for the design of this type of pavement is shown in Figure 12.

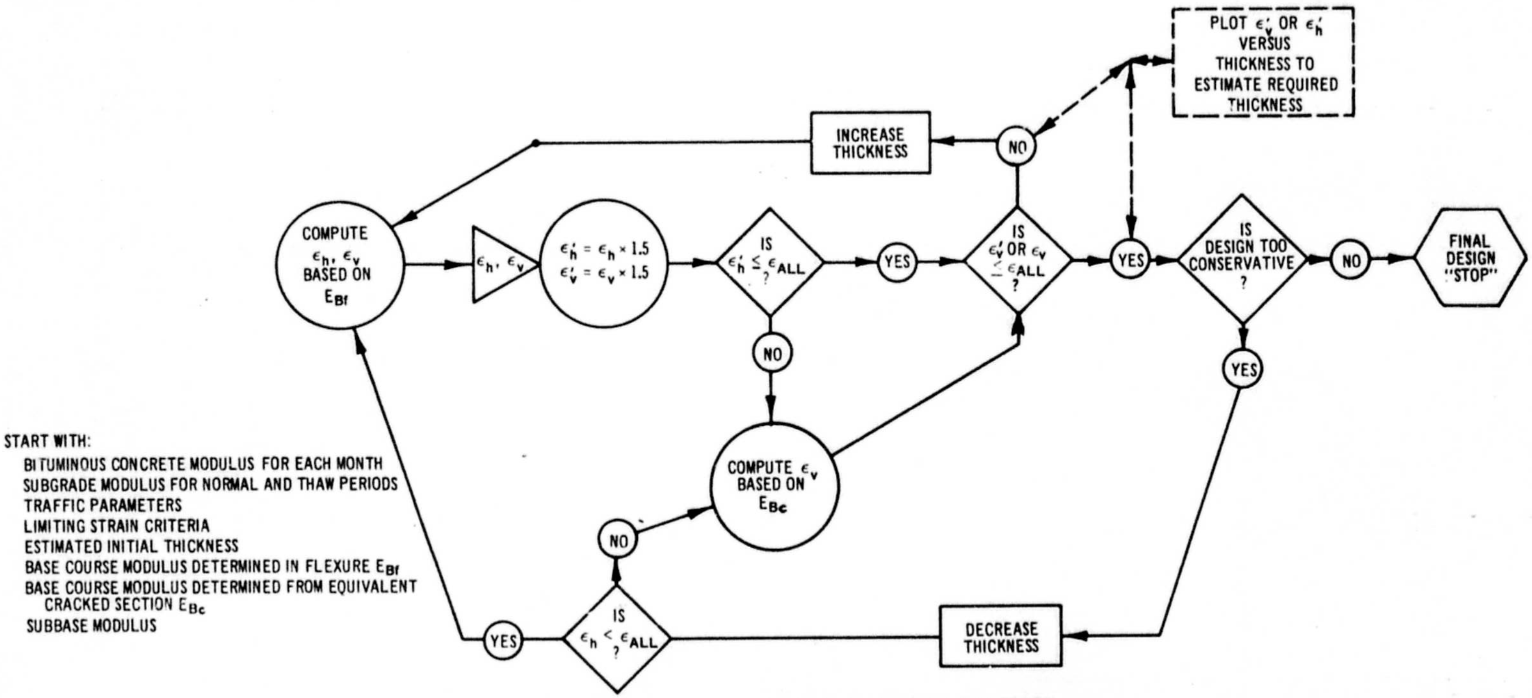
CHEMICALLY STABILIZED PAVEMENT WITH STABILIZED BASE AND SUBBASE COURSES

This type of pavement is handled in a manner almost identical with that for a pavement with a stabilized base. If the base is a bituminous-stabilized material, then the cumulative damage procedure must be employed to determine if the subbase will crack. If the analysis indicates that the subbase will crack due to loading, an equivalent cracked section modulus is determined from Figure 11, and the pavement is treated as a bituminous concrete pavement.

If both the base and subbase courses are stabilized, then both layers must be checked for cracking. A conservative approach is taken by checking for cracking of one layer by considering the other stabilized layer as cracked and having a reduced modulus. The vertical compressive strain at the top of the subgrade is computed using the flexural modulus or the reduced modulus, as appropriate. If either of the two layers is considered uncracked, then the computed subgrade strain is multiplied by 1.5 in order to account for shrinkage cracks which will exist. The basic flow diagram for this type of pavement is shown in Figure 13.

THICKNESS MODIFICATION SUBSYSTEM

If in the performance model it is found that the pavement under consideration does not meet the performance criteria, then it will be necessary to adjust the pavement thickness or the quality of materials and recycle the new design through the system. No specific procedures are advocated for making the modification, although, if the subgrade strain criteria are controlling, the modification will normally involve increasing the thickness of the subbase. When the horizontal tensile strain at the bottom of the bituminous concrete surface course is



NOTE: IF FROST IS A CONSIDERATION, IT IS RECOMMENDED THAT THE SUBGRADE MODULUS DURING THE THAW PERIOD BE USED IN THE DESIGN OF THIS TYPE OF PAVEMENT. THE CUMULATIVE DAMAGE CONCEPT CAN APPLIED IF DESIRED.

ϵ_h' AND ϵ_v' DENOTE STRAINS ADJUSTED FOR SHRINKAGE CRACKING.

Figure 12. Flow diagram of important events for chemically stabilized pavement having stabilized base course and unstabilized subbase course

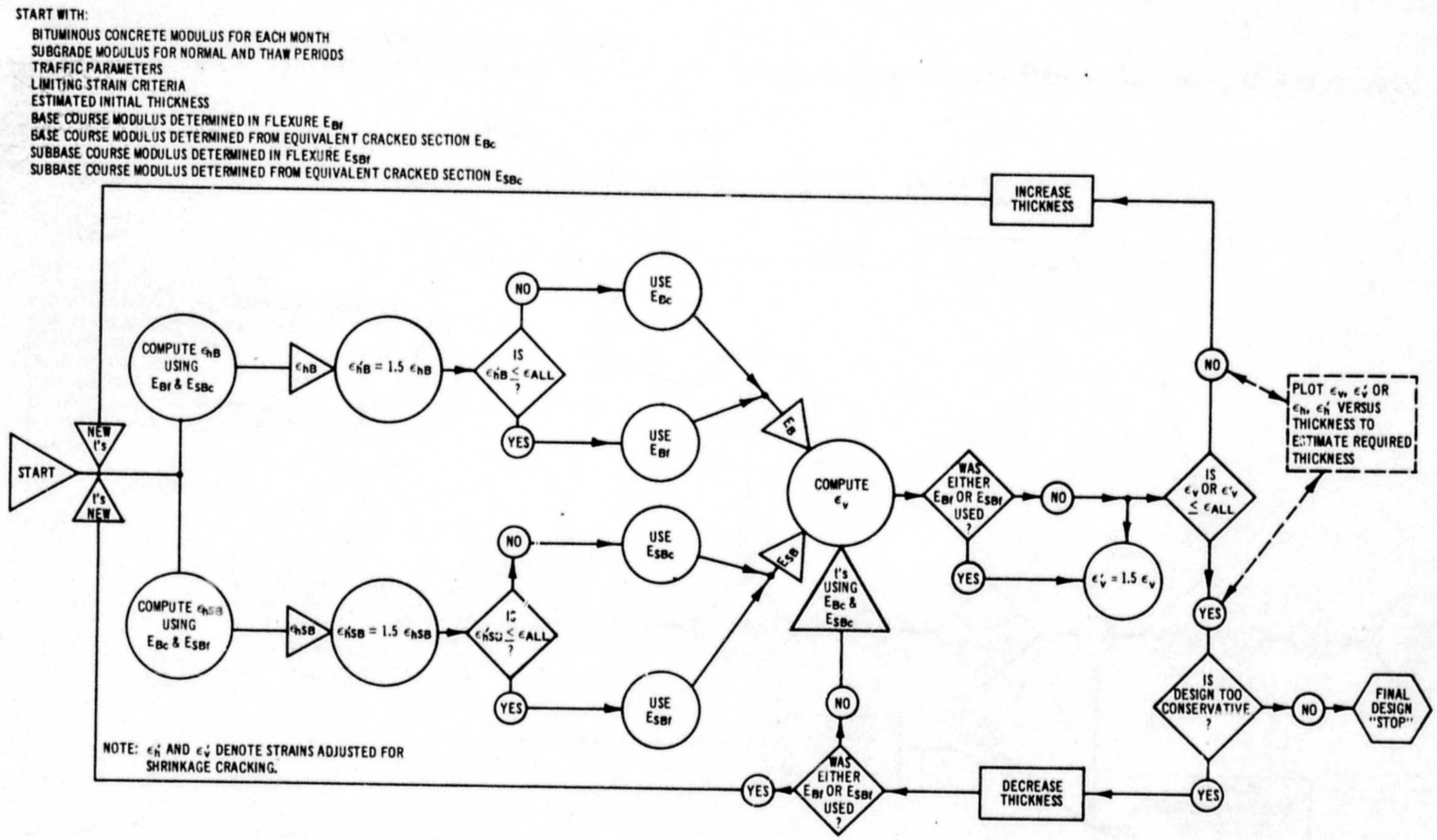


Figure 13. Flow diagram of important events for chemically stabilized pavement having stabilized base and subbase courses

critical, the calculated damage can be reduced by increasing the thickness of this layer, by increasing the stiffness of the base course material through stabilization, or by increasing the thickness of the base course material. Plotting the performance parameters as a function of thickness is an adequate method of estimating the required thickness. Such plots can also indicate if a particular design is overly conservative and if there is a need for reducing the thickness. This technique of modifying thicknesses is indicated in the flow diagrams of the performance models.

DEVELOPMENT OF THE DESIGN CRITERIA

The method of design requires limiting criteria for vertical compressive strain at the top of the subgrade and for horizontal tensile strain at the bottom of bituminous concrete and stabilized layers. The criteria can be developed in one of two ways: either directly from laboratory testing of material specimens or by the analysis of pavement sections of known performance. The development of criteria from pavement sections of known performance has the advantage of indirectly correcting for assumptions necessary to obtain a manageable design system. Unfortunately, data are not always available for establishing all design criteria in this manner, and the costs of generating new data to cover all design situations can be prohibitive. In cases in which performance data are not available, laboratory test data must be used, and assumptions as to the laboratory behavior of materials must be made. In the development of the design procedure presented in this report, conventionally designed pavement sections were used to develop the subgrade strain criteria, while the results of laboratory testing of pavement materials were used to develop the horizontal tensile strain criteria for the structural layers.

SUBGRADE STRAIN CRITERIA

In the development of the subgrade strain criteria from the pavement sections, it was necessary inasmuch as possible to use the same procedures and techniques that would be used in the design system. Thus, the same assumptions were made for the analysis of the pavement sections as would be made in the design of a pavement. Basically these assumptions are:

- a. Traffic can be represented in terms of the number of operations of the fully loaded design aircraft.
- b. Loads are essentially static, and the load on each tire is circular and uniform.
- c. The pavement is a linear elastic layered system with full friction between interfaces.
- d. The bottom layer is of infinite thickness.

e. The deformation characteristics of the pavement materials can be represented by the modulus of elasticity and Poisson's ratio as determined in a repeated load test.

ANALYTICAL RESPONSE MODEL

The analytical response model used in the development of the subgrade strain criteria was a modified version of the CHEVRON computer program.[2] The modification involved revising the input formats and adapting the program to compute response to multiple-wheel loadings and to determine the principal stresses and strains. To determine the total vertical stress or strain at a given point due to a number of wheels, the adaptation is fairly simple in that the effects of each wheel can be added directly. However, if the analysis requires the complete state of stress or strain at a given point, then rotation of stress and strain is necessary, which complicates the computations. The modified version of the CHEVRON computer program is called CHEVIT. The program accomplishes the rotation and superpositions necessary for computation of the complete state of stress and strain at any point in a pavement subjected to multiple-wheel loading. The input form and a listing of the program are presented in Appendix J.

PAVEMENT SECTIONS

The subgrade strain criteria were developed based on data from conventionally designed pavement sections for which a performance life could be assumed. In the development of the criteria, it was desired to use a group of pavement sections which covered a range of design conditions. The design parameters which were to be varied were the subgrade modulus, the design aircraft, and the number of load repetitions. The variations and number of pavement sections required precluded the direct use of test section data. However, since the present CE and FAA thickness design criteria represent a statistical treatment of test section data, it was possible to use the CE and FAA procedures to generate idealized pavement sections. For various loadings of aircraft with single-wheel, dual-wheel, or dual-tandem gears, this procedure was used to generate pavement sections which would perform satisfactorily at 1,200, 6,000, and 25,000 annual departures on 3-, 6-, 10-, and 20-CBR subgrades.

MATERIAL CHARACTERIZATION

In the development of the subgrade strain criteria for design purposes, it was also necessary to use characterization parameters which would be consistent with those of the final design procedure. The parameters for each pavement material used in the development of the subgrade strain criteria are described in the following paragraphs.

Bituminous Concrete. The pavement sections being considered had relatively thin surface courses; therefore, the modulus values selected for the bituminous concrete would, within reason, have little effect on the computed subgrade strains. Since the majority of the test sections represented by the present CE and FAA design procedures were constructed and tested at WES, it seemed appropriate to use a modulus value representative of bituminous concrete in a warm climate. Also, it was believed that the value should be a reasonable one for design purposes. Izatt, Lettier, and Taylor[1] have suggested that for design purposes the modulus during warm weather should range from 150,000 to 200,000 psi. Therefore, a modulus of 200,000 psi was selected as being a conservative value for bituminous concrete for the purposes of developing subgrade strain criteria.

Poisson's ratio for bituminous concrete approaches 0.5 as the modulus decreases; Kingham and Kallas[32] used 0.45 for Poisson's ratio when the bituminous concrete modulus was below 500,000 psi. Since the subgrade strain is very insensitive to the value of Poisson's ratio of the bituminous concrete and for simplicity in this study, a value of 0.5 was selected.

Granular Base and Subbase Courses. Granular materials used in the base and subbase courses are more difficult to characterize, particularly since the characterization technique used must also be applicable to the design procedure. The modulus of granular materials is primarily dependent on the state of stress, quality of materials, degree of compaction, stress history, and moisture conditions. The fact that the modulus is dependent on the state of stress implies that the stiffness of a granular material within a given pavement will vary depending upon the loading applied and the subgrade condition.

Although much work has been conducted recently in an effort to characterize granular materials, nearly all of this work has been in connection with highway pavements. In a nonlinear finite element analysis, Barker[33] characterized granular materials as a function of confining pressure σ_3 . He showed that for wheel loads greater than 30 kips the response of the pavement system was governed by a minimum modulus used when a negative (tensile) value of σ_3 was computed. In a similar study, Chou, Hutchinson, and Ulery[34] attempted to circumvent the problem by characterizing the material as a function of θ , where θ is the sum of the principal stresses σ_1 , σ_2 , and σ_3 . This procedure resulted in less drastic variations in modulus values but still resulted in negative stresses being computed for the confining stresses. The question is thus raised as to how granular material is able to exhibit a positive modulus value when the confining stress is negative.

Kirwan, Glynn, and Bonner[35] used a finite element computer code which, when tensile stresses are encountered, applies balancing nodal forces to eliminate the tensile stresses. Unreported work by the authors with this computer code in the analysis of airport pavements has indicated better agreement with measured values than that reported by Barker,[33] Chou, Hutchinson, and Ulery,[34] and Barker, Brabston, and Townsend[36] but at the same time has demonstrated the complexity which can result from attempts at a theoretical characterization of granular materials. These nonlinear finite element studies of airport pavements have led the authors to the conclusion that the state-of-the-art required, for this initial design system, a simpler approach to the handling of granular materials.

Based on this conclusion, it was decided to use a procedure to determine a quasi-modulus based on an anticipated state of stress. Izatt, Lettier, and Taylor[1] used this approach in which the modulus of the granular material is a function of the layer thickness and modulus of the subgrade. The relationship is shown in Figure 14. In this

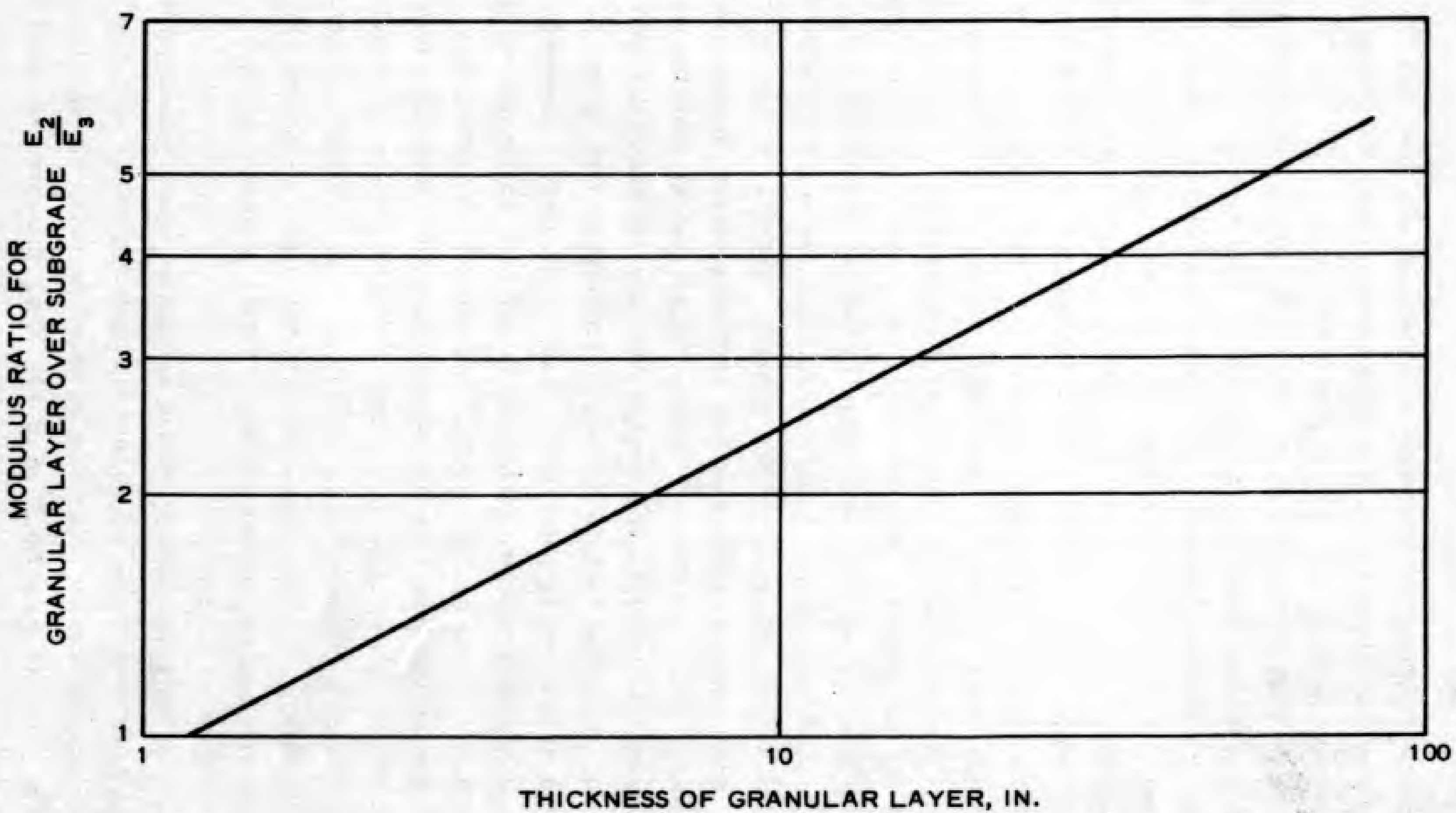


Figure 14. Relationship between modulus ratio and granular layer thickness (after Izatt, Lettier, and Taylor[1])

procedure, the granular material is considered to be one layer, and an average modulus is assigned for the entire layer. All of the previously mentioned finite element analyses of pavements have shown large variations in the modulus from the top to the bottom of the granular layer. Although the use of an average modulus does not significantly affect the value of the computed subgrade strain, the horizontal tensile strain at the bottom of the bituminous concrete has been found to be highly sensitive to the modulus of the material in the upper portion of the granular layer. For this reason, it was decided that for thicker granular layers such as would be encountered in airport pavements, the modulus values should reflect the variation from top to bottom.

To provide for the variation, the granular layers were divided into sublayers for which the modulus of each sublayer would be a function of the sublayer thickness and the modulus of the material below the sublayer. In developing the subgrade strain criteria, it was necessary to develop relationships for high-quality base and subbase course materials. The nonlinear finite element analyses of pavement sections and the relationship presented by Izatt, Lettier, and Taylor[1] were used as the basis for developing these relationships. Based on the relationship shown in Figure 14 for a subgrade modulus of 6000 psi, the relationships shown in Figure 15 between modulus ratio and thickness were assumed.

It was considered that each of the materials had a practical limiting modulus, i.e., regardless of how thick the layer or how stiff the layer beneath, the modulus of the material would approach some limiting value. From the results of the finite element analyses, limiting values of 100,000 and 40,000 psi were chosen for base and subbase course materials, respectively. The graph shown in Figure 15 along with the limiting modulus values provided two points from which the relationships shown in Figure 16 were developed. From Figure 16, the relationships shown in Figure 17 were developed by which the modulus of a sublayer can be determined directly from the sublayer thicknesses and from the modulus of the underlying sublayer. Figure 17 was the basis for determining the modulus values of the granular base and subbase course materials of the pavement sections used in the development of the subgrade strain

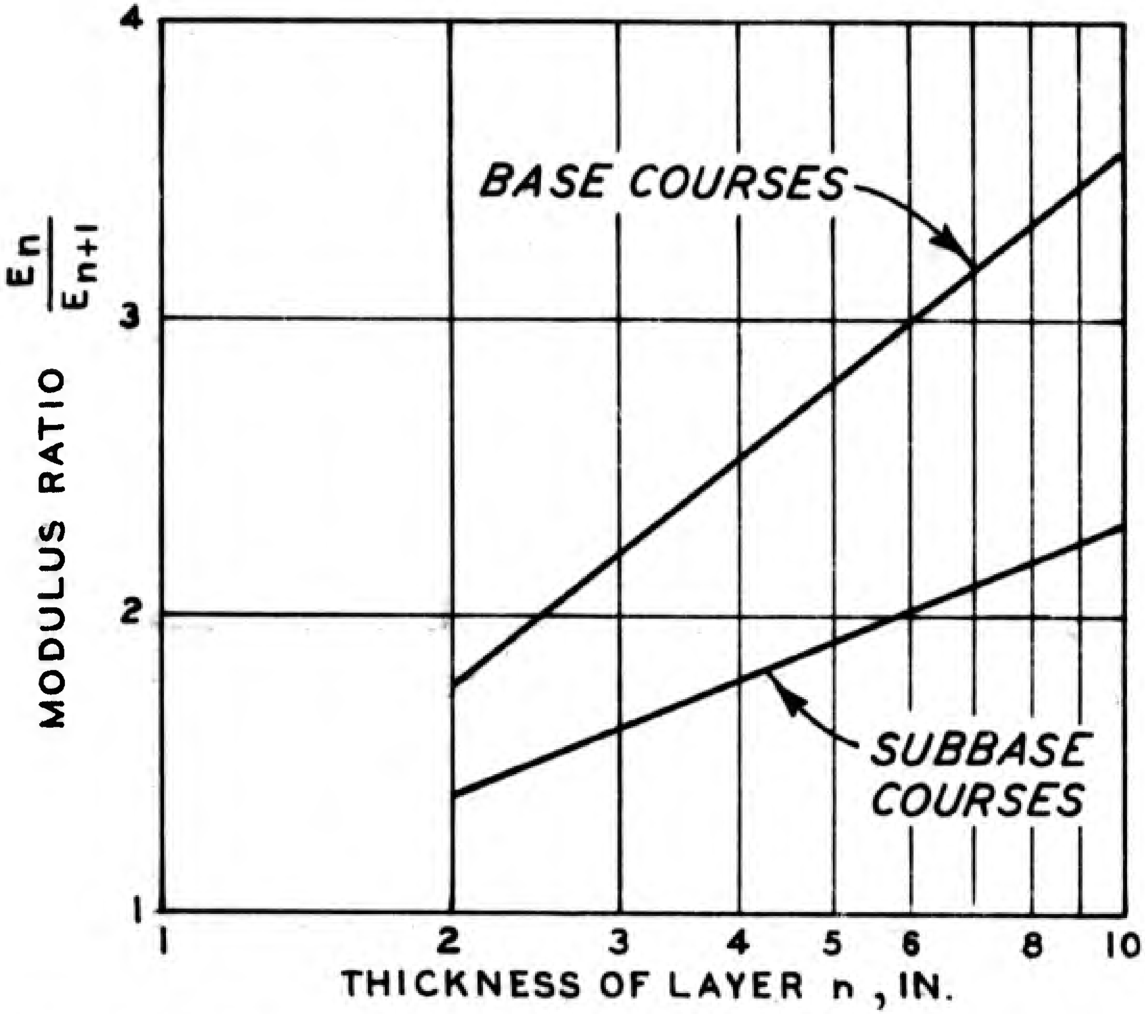


Figure 15. Relationships between modulus ratio and thickness of layer n for a modulus of layer n + 1 of 6000 psi

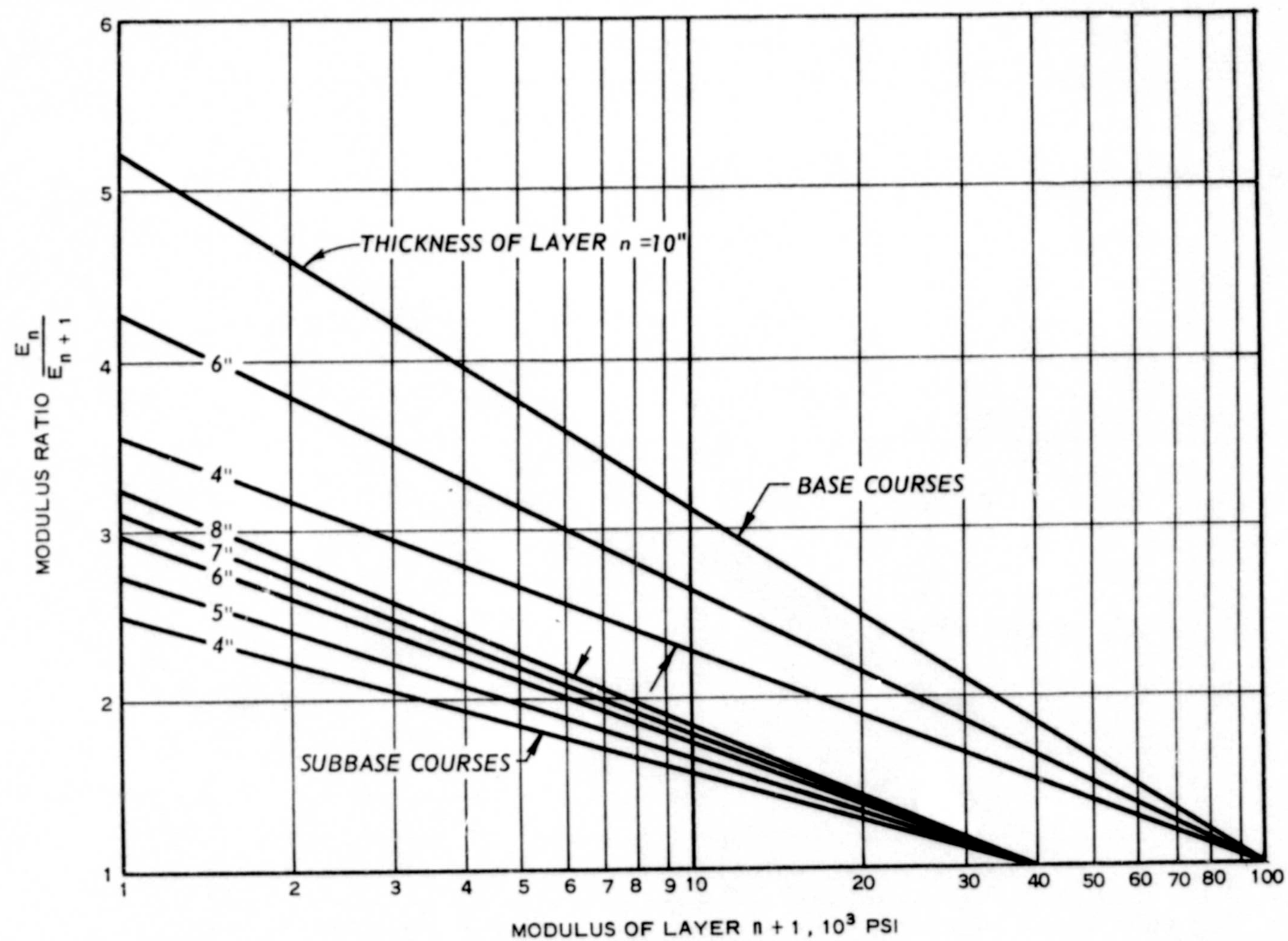


Figure 16. Relationships between modulus ratio and modulus of layer n + 1 for various thicknesses of layer n

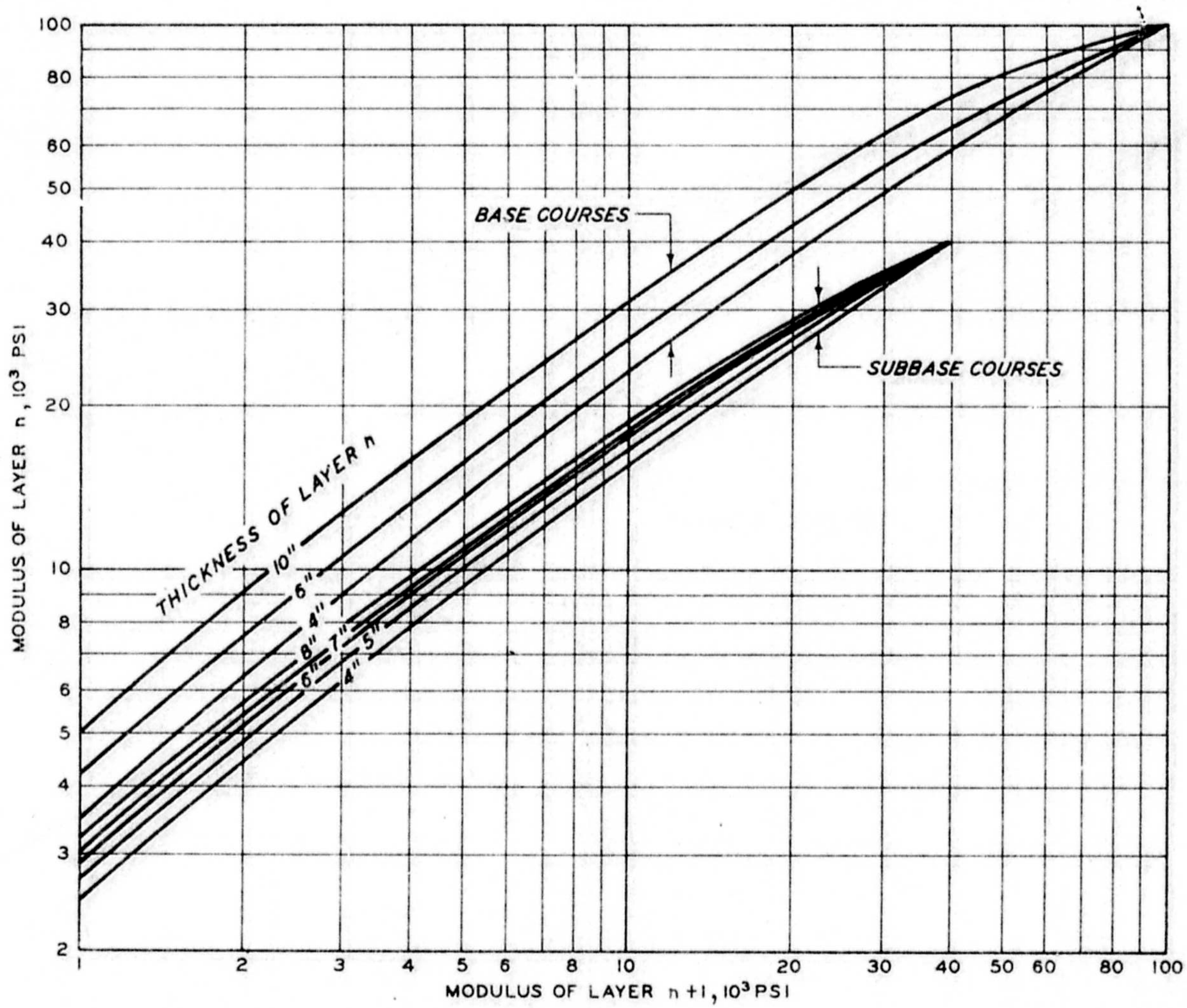


Figure 17. Relationships between modulus of layer n and modulus of layer n + 1 for various thicknesses of layer n

criteria. Analytically, the relationships between E_n , E_{n+1} , and thickness t shown in Figure 17 can be represented in terms of material constants a , b , c , d , and e by the equation

$$E_n = E_{n+1}\left(a + \left(\frac{a-1}{\log \frac{b}{e}} \log \frac{t}{b}\right) + \left\{\left(\log \frac{c}{E_{n+1}}\right)\left[\frac{(a-1)\left(1 + \frac{\log \frac{t}{b}}{\log \frac{b}{e}}\right)}{\log \frac{d}{c}}\right]\right\}\right) \tag{9}$$

where the material constants are defined as follows:

a = ratio of E_n to E_{n+1} for a layer with thickness b over a material having a modulus of c

b = thickness of the layer having the E_n to E_{n+1} ratio of a

c = modulus of the layer beneath the layer having the E_n to E_{n+1} ratio of a

d = maximum limiting modulus for the particular material

e = layer thickness for which the modulus ratio is unity

If the following definitions are assumed:

$$X = \frac{a-1}{\log \frac{b}{e}}$$

$$Y = \log \frac{d}{e}$$

$$T = \frac{X}{Y}$$

$$R = a - X \log b + \frac{a-1}{Y} \log c - T \log c \log b$$

$$S = X + T \log c$$

$$W = T \log b - \frac{a-1}{Y}$$

then Equation 9 can be rewritten in terms of new material constants R , S , T , and W as

$$E_n = E_{n+1} (R + S \log t - T \log t \log E_{n+1} + W \log E_r) \quad (10)$$

For characterization of the subbase course, it was assumed that a = 2, b = 6 in., c = 600 psi, d = 400 psi, and e = 1 in.; therefore, R = 1, S = 7.18, T = 1.56, and W = 0. The resulting equation for determining the subbase modulus is

$$E_n = E_{n+1} (1 + 7.18 \log t - 1.56 \log E_{n+1} \log t)$$

For base course materials, it was assumed that a = 3, b = 6 in., c = 600 psi, d = 100,000 psi, and e = 1 in.; therefore, R = 1, S = 10.52, T = 2.10, and W = 0 with a resulting equation of

$$E_n = E_{n+1} (1 + 10.52 \log t - 2.10 \log E_{n+1} \log t)$$

Subgrade. The pavement sections represented which perform satisfactorily on subgrades ranging in quality as presently defined by the CE and FAA. In the design procedure presented in this report, the resilient modulus is used as the basis of measuring the quality of the subgrade. It is evident, therefore, that in this design procedure, the thickness of material required to protect the subgrade is related to both the subgrade CBR and the resilient modulus. The implication of this relationship is that the subgrade CBR is directly related to the resilient modulus, which may or may not be true for all subgrade soils but certainly should be true for the subgrades of the test sections from which the present CE and FAA design procedures were developed. Heukelom and Klomp[30] presented such a relationship between dynamic modulus and CBR in the form $E = 1500 \times CBR$. Green and Hall[37] proposed a slightly different relationship (Figure 18) which indicated a higher modulus for lower CBR's and a lower modulus for higher CBR's. A comparison of the relationships as presented by Green and Hall[37] is shown in Figure 19.

In view of the previous wide use of the relationship developed by Heukelom and Klomp, it was selected as a method of estimating the resilient modulus for the pavement sections. A check of the relationship was possible from the data presented in Figure 20. These data, which were obtained from undisturbed samples of a 4-CBR heavy clay (E-11, CH) used in WES test sections, represent results of resilient modulus tests of

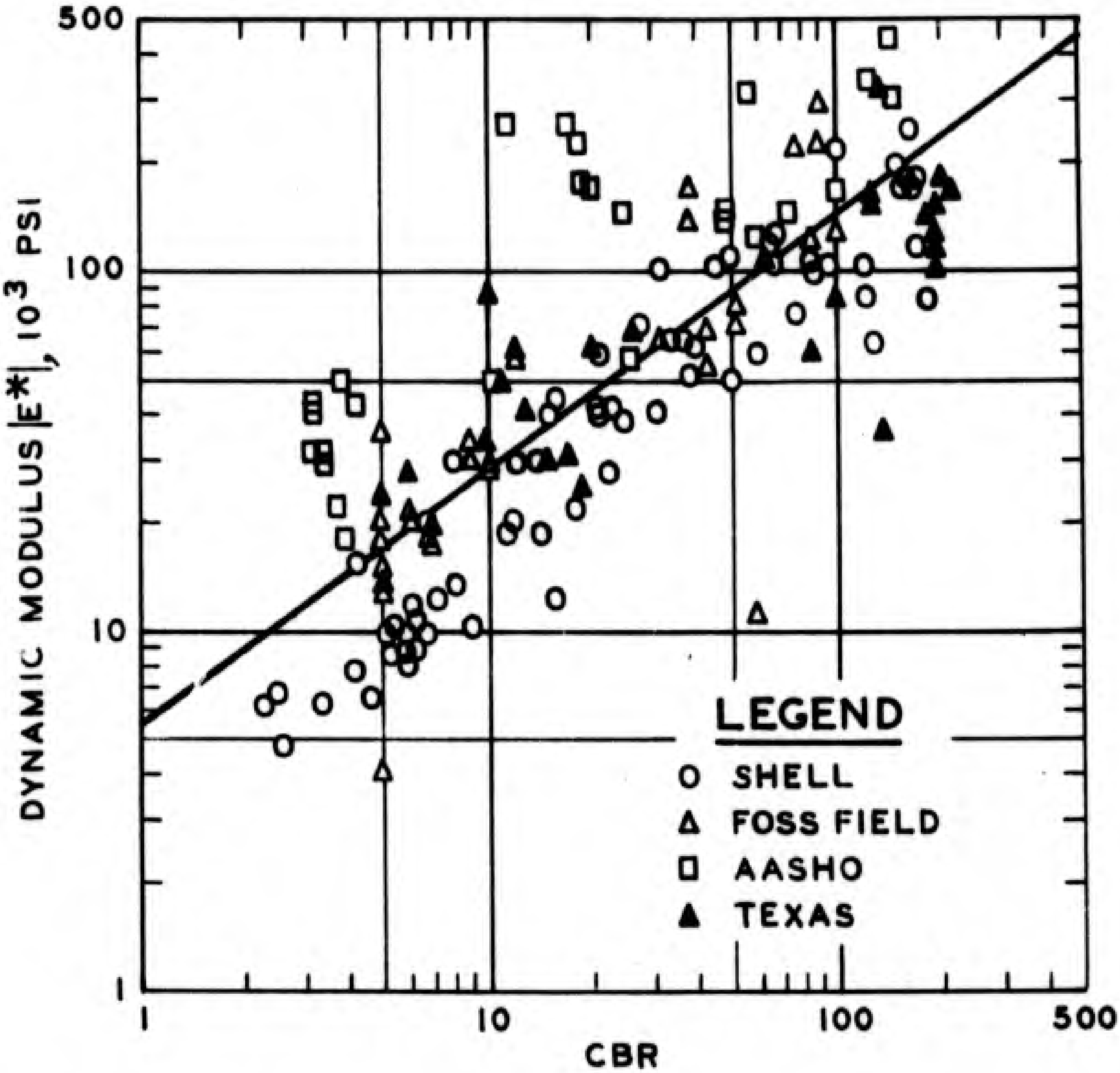


Figure 18. Relationship between dynamic modulus and CBR; WES correlation (after Green and Hall[37])

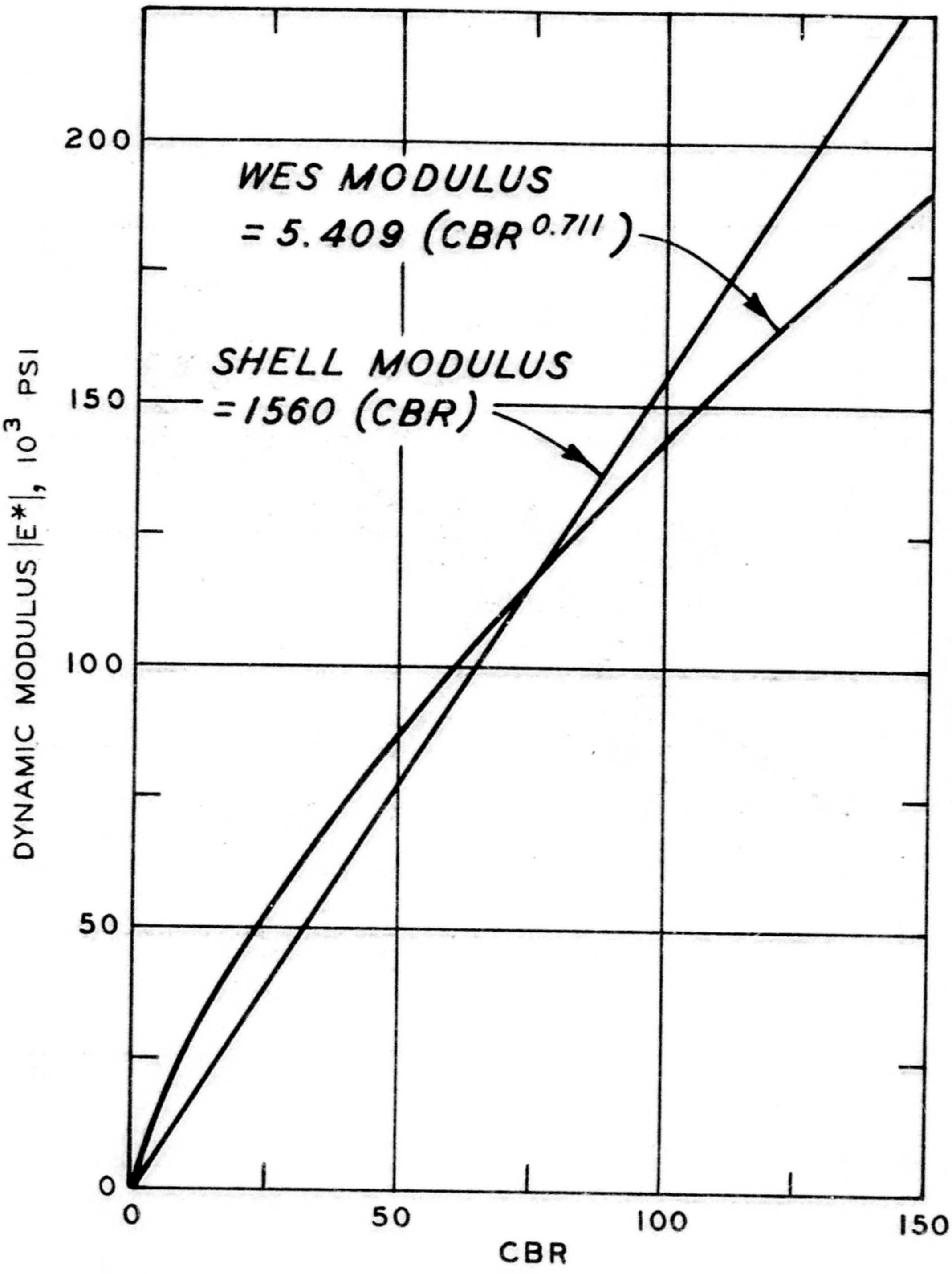


Figure 19. Relationships between dynamic modulus and CBR; comparison of WES and Shell correlations (after Green and Hall[37])

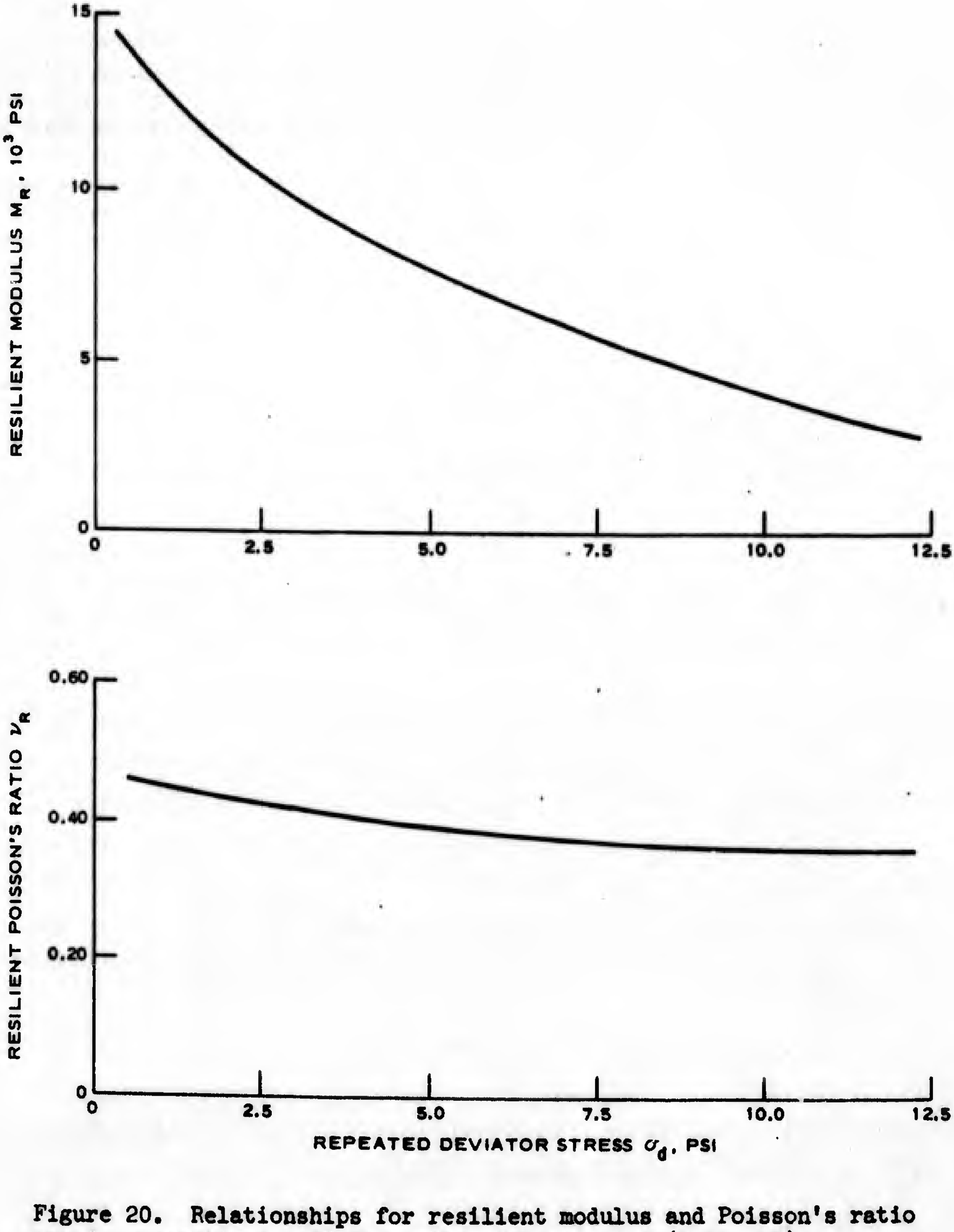


Figure 20. Relationships for resilient modulus and Poisson's ratio based on resilient triaxial tests of heavy clay (E-11, CH)

the type subgrade on which the present FAA and CE design procedures are based. For this subgrade, the stress at the top of the subgrade for a well designed pavement would be on the order of 7 psi. From Figure 20, the resilient modulus would be approximately 6000 psi, or 1500 CBR for a stress of 7 psi.

The data in Figure 20 also show that a value of Poisson's ratio of 0.4 would be appropriate. For various quality subgrades, the value of Poisson's ratio would be expected to range between 0.25 and 0.5. For the purpose of developing the subgrade strain criteria, a constant Poisson's ratio of 0.4 was chosen.

STRAIN COMPUTATIONS

With the methodology established for characterizing the materials, the computations were made to determine the subgrade strain. For example, strain was computed for a pavement section required for 25,000 annual departures of a 50-kip aircraft having a single-wheel gear on a 3-CBR subgrade. The pavement section would consist of a 3-in. bituminous concrete surface course, a 6-in. base course, and a 27-in. subbase course over the subgrade. According to the methodology for determining material properties, the section to be analyzed would be as shown in Figure 21. The aircraft load data would be a single-wheel load of 21.375 kips (gross aircraft loading of 50 kips) and a tire contact pressure of 90 psi. Using the CHEVIT computer program with the above data, the maximum vertical compressive strain at the top of the subgrade was computed. For this section, the computed strain was 0.648×10^{-3} in./in.

In a similar manner, computations were made for other sections involving different subgrade strengths, traffic levels, and aircraft loads. From these data, the subgrade strain criteria shown in Figure 8 were developed. Figure 7 presents these strain criteria in a form that can be more easily used for determining the number of allowable strain repetitions N for input to the cumulative damage relations.

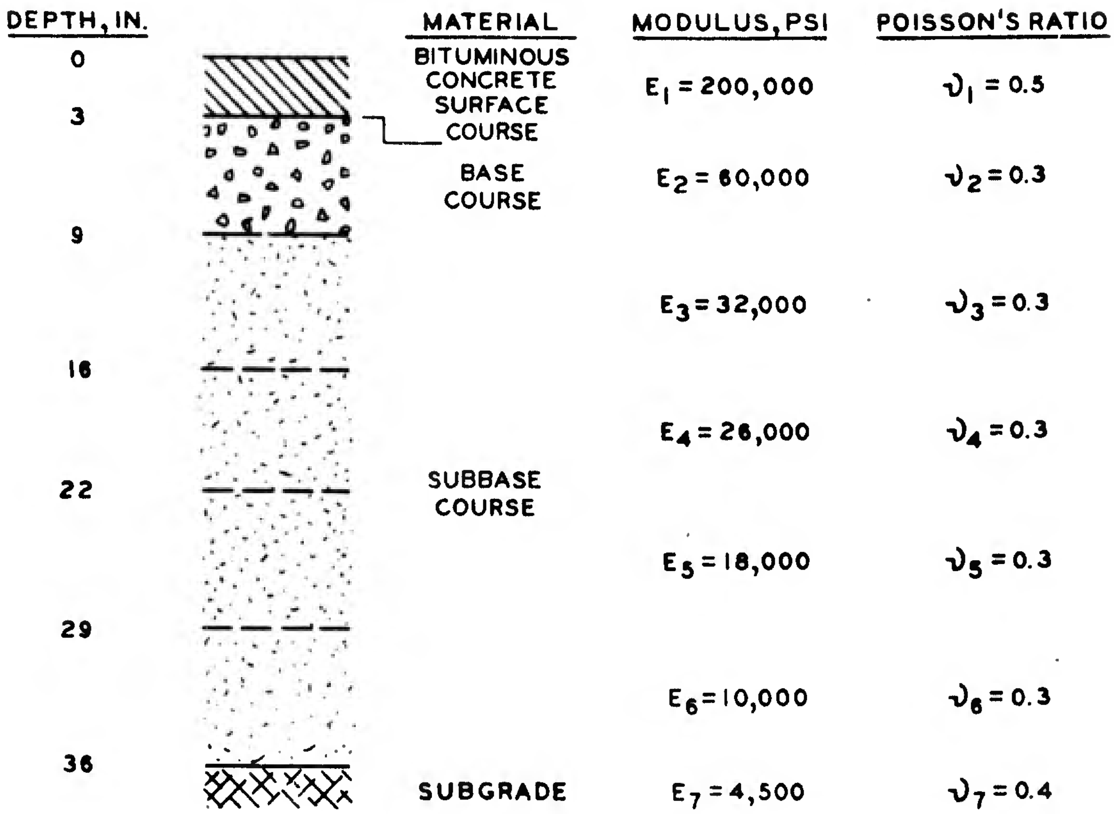


Figure 21. Idealized pavement section for 25,000 annual departures of 50-kip, single-wheel aircraft on 3-CBR subgrade

VERIFICATION OF SUBGRADE STRAIN CRITERIA

The WES test section data used in the verification are presented in Ahlvin et al.[38] and Burns et al.[39] The purpose of the study described in Ahlvin et al. was to validate present criteria or establish new criteria for the evaluation and design of pavements subjected to multiple-wheel heavy gear load (MWHGL) aircraft. The MWHGL flexible pavement test section contained 5 test items of varying thicknesses of conventional flexible pavement construction. For verification of the criteria presented in this report, items 1, 2, 3, and 5 were used. Item 4 was omitted because the structure and performance of this item were essentially the same as those of item 3. These four items represent pavement thicknesses of 15, 24, 33, and 42 in. above the subgrade. Test loadings considered in the verification were single-wheel loads of 30 and 50 kips, a 12-wheel C-5A loading of 360 kips, and dual-tandem loadings of 200 and 240 kips. The study described in Burns et al.[39] was designed to evaluate the performance of pavement sections having stabilized layers. The flexible pavement test section, referenced as the "structural layers" test section, contained 5 test items, the first four of which contained stabilized layers. Item 5 was of conventional flexible pavement construction of the same thickness as item 5 of the MWHGL test section and was therefore not considered in this study. The traffic on item 4 resulted in early failure which was judged to be due to shear failure in the stabilized material; thus, this item could not be used in verification of the subgrade strain criteria. The trafficking of the remaining three items with a dual-tandem gear loaded at 200 and 240 kips provided data for additional verification of the criteria. Layouts of the two test sections are shown in Figures 22 and 23.

For computation of strains, the relationship presented in Figure 20 was used to estimate the subgrade modulus. The technique used was to first estimate a modulus value for a particular loading on the section, compute the stress at the top of the subgrade, and then adjust the modulus based on the computed stress. In this manner the subgrade modulus used was compatible with the relationship in Figure 20 and the

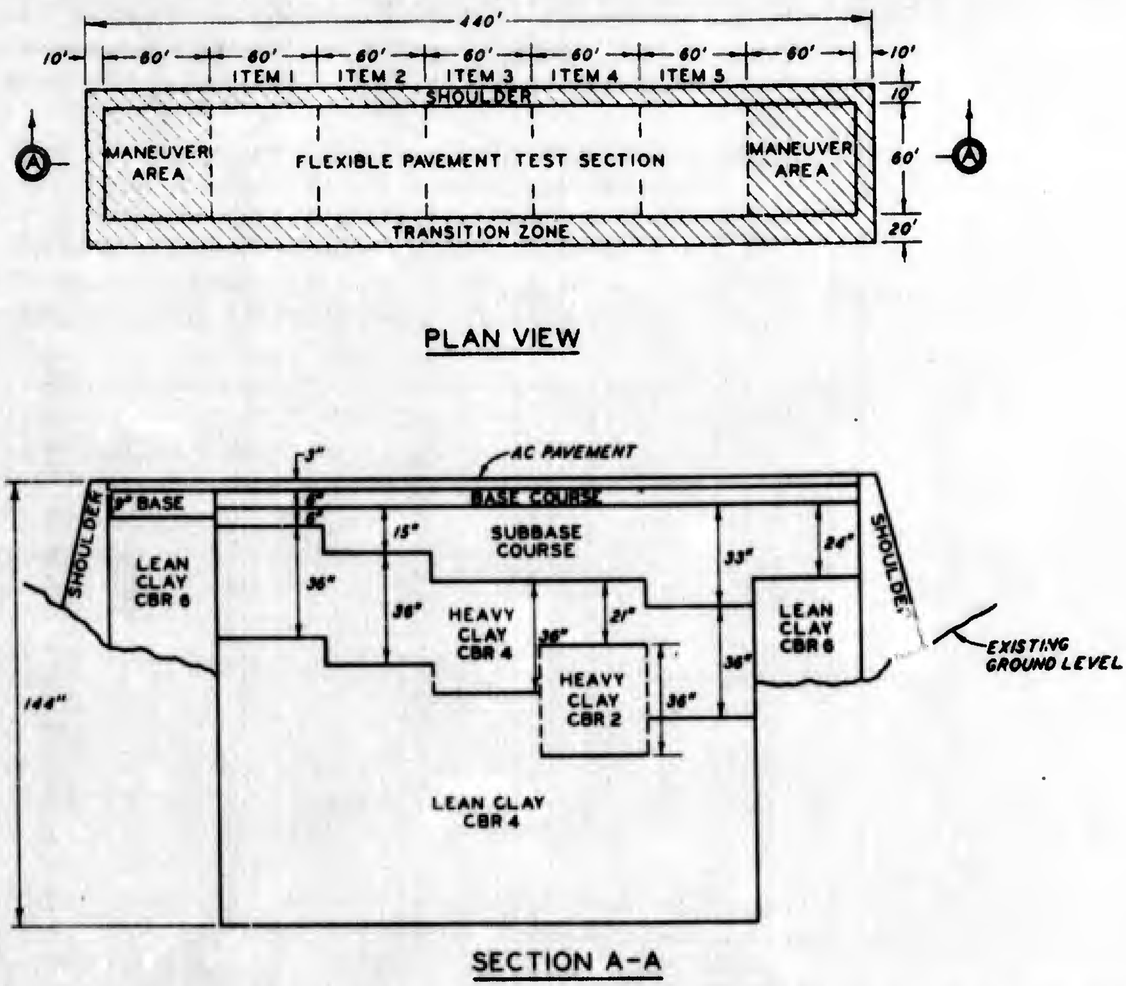


Figure 22. Layout of MWHGL flexible pavement test section (after Ahlvin et al.[38])

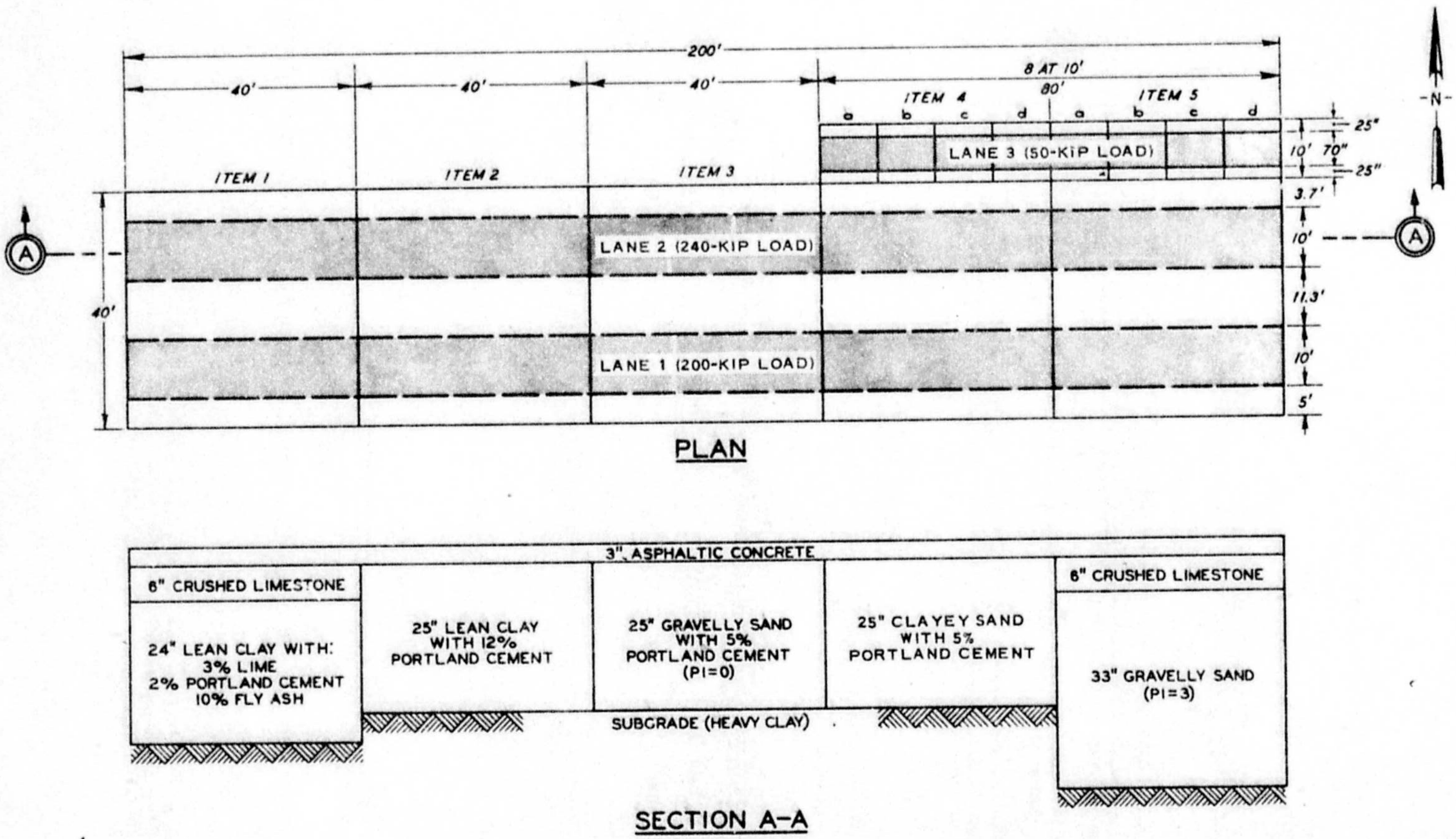


Figure 23. Layout of structural layers flexible pavement test section (after Burns et al.[39])

computed stress at the top of the subgrade. The modulus values for the granular base and subbase courses were determined as outlined in Appendix G. The modulus values for the stabilized materials were determined from Figure 11 based on unconfined compressive strengths of 250, 500, and 800 psi for lean clay stabilized with lime, fly ash, and portland cement; cement-stabilized lean clay; and cement-stabilized gravelly sand, respectively. These data yielded modulus values of 30,000, 60,000, and 100,000 psi for the stabilized materials of items 1, 2, and 3, respectively, of the structural layers test section.

Test section data used in the verification are presented in Table 3. From these data, the plot in Figure 24 was developed. This plot shows the relationship between passes of the test gear and computed subgrade strain. Shown along with these data are the subgrade strain criteria as developed from the idealized pavement sections. Although none of the test data extend to the traffic level of the criteria, a logical extrapolation of the criteria can be made which could also represent a criteria curve drawn for the test section data. It should be noted in the figure that points 1, 4, and 3 (considering that point 3 represents a nonfailure) deviate the farthest from the extrapolated criteria curve. All three of these points are for single-wheel traffic; thus, there may be a discrepancy in the analysis of multiple- and single-wheel test data. This discrepancy would occur, at least in part, due to the assumption that each pass constitutes a stress repetition. The items for which single-wheel traffic data were available were relatively thin, causing a narrow width of subgrade to be severely strained. The multiple-wheel data are available for thicker items in which a wider section of the subgrade would be affected. Thus, the use of passes to represent strain repetitions is more appropriate for multiple-wheel than single-wheel gears.

Although the limited comparisons presented do not represent a complete verification of the subgrade strain criteria, the comparisons do lend credibility to the criteria. Certainly, there are no data within the analysis which contradict the criteria.

Table 3

MWHGL[38] and Structural Layers[39] Test Section Data

Test Point No.	Item No.	Load kips	Type of Assembly	No. of Passes to Failure	Subgrade Modulus psi	Subgrade Strain 10^{-3} in./in.	Stabilized Layer Modulus psi
			MWHGL Test Section				
1	1	30	Single-wheel	636	4000 3700	4.8	NA
2	1	180	12-wheel	11	2500 4000	5.0	NA
3	2	30	Single-wheel	2,385*	4500	2.1	NA
4	2	50	Single-wheel	1,063	4500	3.5	NA
5	2	180	12-wheel	275	4500	2.5	NA
6	3	180	12-wheel	2,062	5000	1.6	NA
7	3	240	Dual-wheel	60	3000	4.2	NA
8	5	180	12-wheel	5,293*	6000	1.1	NA
9	5	240	Dual-wheel	420	3500	2.7	NA
			Structural Layers Test Section				
10	1	200	Dual-tandem	5,490	5300	2.0	30,000
11	1	240	↓	900	4800	2.5	30,000
12	2	200	↓	5,490	5350	1.9	60,000
13	2	240	↓	510	4800	2.4	60,000
14	3	200	↓	11,730	6200	1.5	100,000
15	3	240	↓	900	5700	1.8	100,000

* Failure did not occur at this test point.

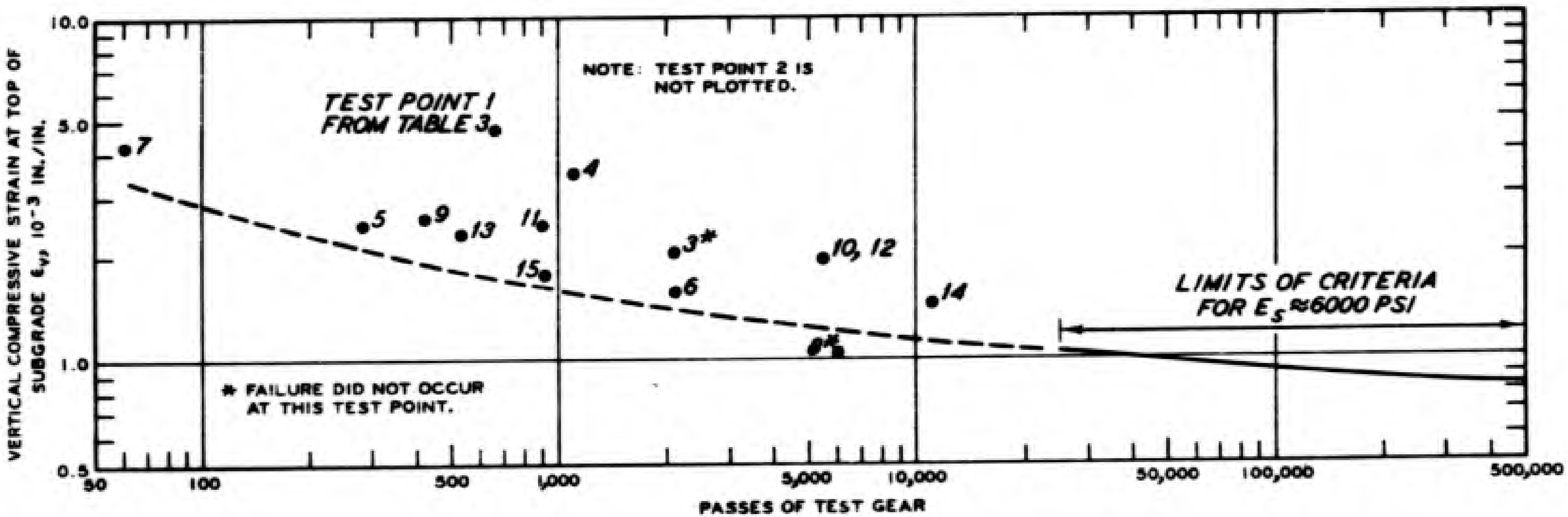


Figure 24. Relationship between computed subgrade strain and passes of test gear

SUBGRADE STRAIN CRITERIA COMPARISONS

Numerous researchers have developed strain criteria for design of flexible pavements. Among these are Witczak,[4] Edwards and Valkering,[40] Dorman and Metcalf,[41] Finn, Nair, and Monismith,[42] Brabston, Barker, and Harvey,[7] and Chou, Hutchinson, and Ulery.[34] In addition, Peattie[43] has presented criteria based on subgrade stress. A comparison of these criteria is presented in Figure 25. In order to include the criteria of Peattie in the comparison, the subgrade strain was computed by assuming

$$\varepsilon_v = \frac{\sigma}{E_S}$$

where

σ = allowable subgrade stress

E_S = modulus of the subgrade (which is determined from the relationship $E = 1500 \times CBR$)

Peattie's criteria are for 1,000,000 strain repetitions and are a function of the subgrade CBR. Although Edwards and Valkering use the strain criteria of Dorman and Metcalf, they also present strain criteria for 1,000,000 strain repetitions which were developed from conventional flexible pavement sections designed according to the Shell CBR design curves. These strain criteria are given as being between 0.8×10^{-3} and 0.9×10^{-3} in./in., but no reason is given for the range in the criteria. The criteria of Finn, Nair, and Monismith are also presented in a band, but again no explanation is given for the range. The criteria presented by Brabston, Barker, and Harvey are presented as functions of the subgrade modulus and are the only strain criteria so presented.

The first comparison to be made is with the criteria developed by Chou, Hutchinson, and Ulery[34] in which a nonlinear finite element computer program was used in computing the subgrade strain. These criteria were developed from essentially the same test data as were used in the verification of the criteria presented in this report. If the criteria presented by Chou, Hutchinson, and Ulery (Figure 26) are extrapolated to the repetition range of the WES criteria, it can be seen that they would closely match the latter. The curve in Figure 26 was drawn through the

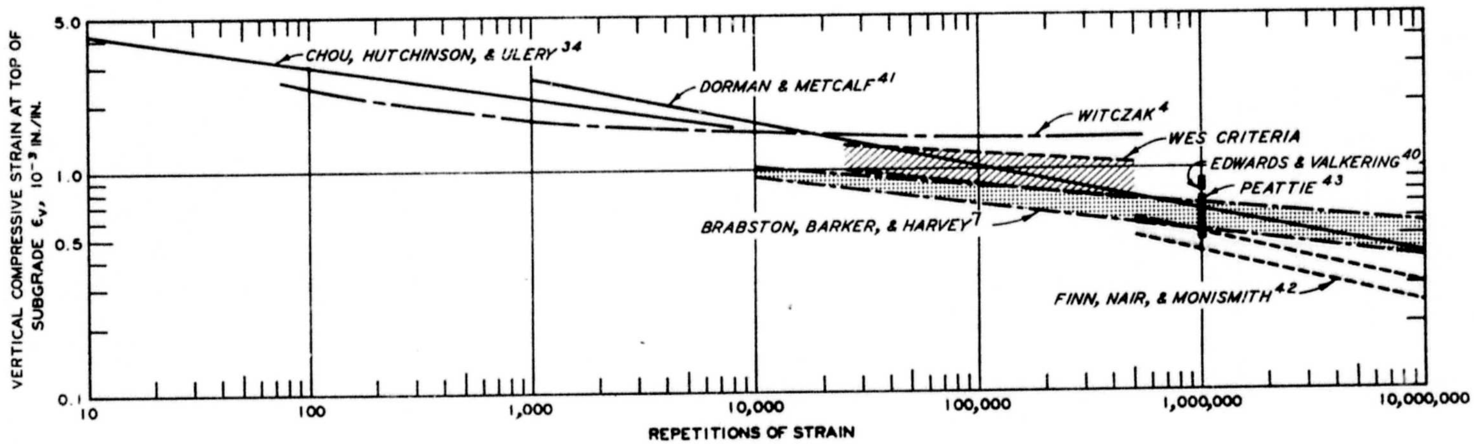


Figure 25. Comparison of subgrade strain criteria

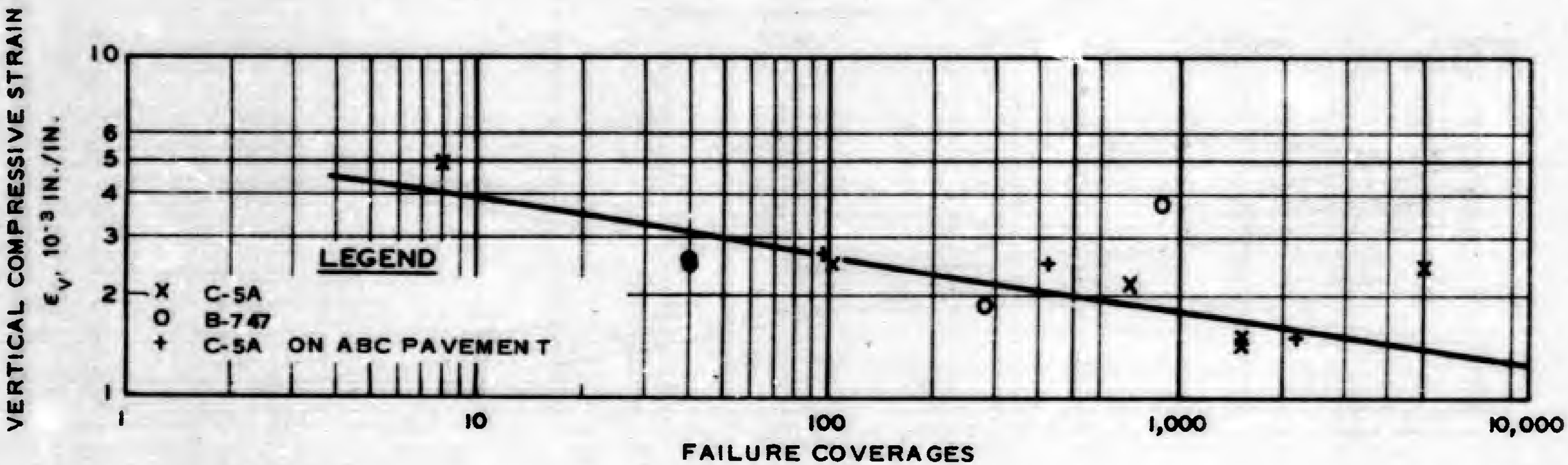


Figure 26. Relationship between subgrade strain and performance under multiple-wheel loads (after Chou, Hutchinson, and Ulery[34])

center of the data points whereas the curve in Figure 24 was drawn so that the data would fall above it. If the curve in Figure 26 was redrawn so that it was positioned below the data points and extrapolated to the repetition range of the WES data, then the criteria would be almost identical with the WES criteria for soils having modulus values appropriate for the 4-CBR subgrade soil.

Other subgrade strain criteria have been presented by Witczak.[4] Although Witczak developed his criteria using a range of assumed asphaltic concrete moduli, the particular curve chosen for comparison was developed for an asphaltic concrete modulus of 200,000 psi (the same as that used in developing the WES criteria). The criteria developed by Witczak indicate larger strain values than the WES criteria and are less sensitive to increases in the number of strain repetitions. These criteria, over the range of strain repetitions covered in this design procedure have an almost constant value. However, it should be noted that a straight-line extrapolation from the initial portion of the criteria (that portion covered by actual test data) would fall within the bounds of the WES criteria.

Other comparisons may be made with criteria developed from road design data. The criteria presented by Brabston, Barker, and Harvey[7] for the design of ABC pavements for military roads were developed in an almost identical manner as the WES criteria, except that a slightly different procedure was used to determine the modulus of the granular materials. These criteria indicate lower strain values than the WES criteria. This trend could be due to the different procedures used for characterizing the granular materials or the different performance criteria for military roads and airport pavements.

Another comparison with road criteria may be made with those developed by Dorman and Metcalf.[41] Although they were developed from road data, it has been suggested that these criteria can also be used for airport pavement design. These criteria, in contrast to those of Witczak, are more sensitive to increases in the number of strain repetitions than the WES criteria. A plot of the Dorman and Metcalf criteria passes diagonally through the WES criteria and thus indicates good agreement, even though Dorman and Metcalf criteria have no dependency on subgrade modulus.

The criteria presented by Finn, Nair, and Monismith[42] are more sensitive to increases in the number of strain repetitions than the WES criteria. When extrapolated to the repetition range of the WES criteria, they fall within and below the WES criteria. The criteria at the higher repetition levels indicate lower strain values than those of Dorman and Metcalf[41] and Brabston, Barker, and Harvey,[7] which would result in much thicker pavement designs at the higher repetition levels.

In comparisons presented by Brabston, Barker, and Harvey[7] of pavement thicknesses determined using different procedures for high strain repetitions levels, the Finn, Nair, and Monismith criteria result in design sections almost twice as thick as those indicated by any other criteria. At low repetitions levels, the pavement thicknesses were almost identical with those determined using the other procedures.

The criteria developed by Edwards and Valkering[40] and Peattie[43] were for 1,000,000 strain repetitions only. Extrapolation of the WES criteria shows that they agree well with those of Edwards and Valkering at 1,000,000 strain repetitions.

The Peattie criteria, which were computed from stress criteria, extend entirely across the criteria band presented by Brabston, Barker, and Harvey for military roads but fall only partially within the extrapolation of the WES criteria.

RELATIONSHIP BETWEEN ALLOWABLE STRAIN AND SUBGRADE MODULUS

Of the criteria reviewed, only the subgrade strain criteria of Brabston, Barker, and Harvey[7] and the stress criteria of Peattie are presented as a function of the subgrade modulus. When strain criteria are determined from stress criteria, it is found that these also are a function of the subgrade modulus. If the WES criteria for 500,000 repetitions are plotted along with the extrapolation of these criteria to 1,000,000 repetitions and the Brabston, Barker, and Harvey[7] and Peattie[43] criteria for 1,000,000 repetitions, a comparison can be made between them on the basis of the relationship between subgrade modulus and allowable strain (Figure 27). The WES criteria are slightly more sensitive to changes in the subgrade modulus than the other criteria. Although the criteria presented by Brabston, Barker, and Harvey and Peattie have almost identical end point values, the Peattie curve is concave upward whereas both of the other curves are concave downward. Considering the assumption made in computing the strain criteria from the stress criteria, this difference may not be significant.

The dependence of the allowable strain on subgrade strength has been the source of some concern since such dependence has not been previously reported. A project was therefore initiated to study the effect of soil strength on the relationship between resilient strain and permanent strain. In this study specimens of a clay soil at four strengths as measured in the soaked CBR test were tested under repeated loadings for which both resilient and permanent strains were measured. Preliminary results from these tests are shown in Figure 28. The data indicate that for a given resilient strain the specimens of the weaker soil display larger permanent strains than those of the stronger soil. The design procedure presented herein, in concept, limits the permanent strain by limiting the resilient strain. Thus, if the preliminary interpretation of these laboratory test results is correct, the allowable resilient strain will have to be less for weaker soils than for stronger soils in order for the permanent strain in both to be the same. A complete analysis of the results of these laboratory tests is in progress and will be reported.

HORIZONTAL TENSILE STRAIN CRITERIA FOR BITUMINOUS CONCRETE

In selection of the strain criteria for bituminous concrete, it was assumed that failure of a pavement occurs at the same time as initial cracking and that the fatigue strength of the material can be evaluated by laboratory testing. It is generally recognized that the fatigue strength of bituminous materials is highly dependent not only on the type of mix but also on the temperature, stress history, and mode of testing. In this study, temperature was considered the most important

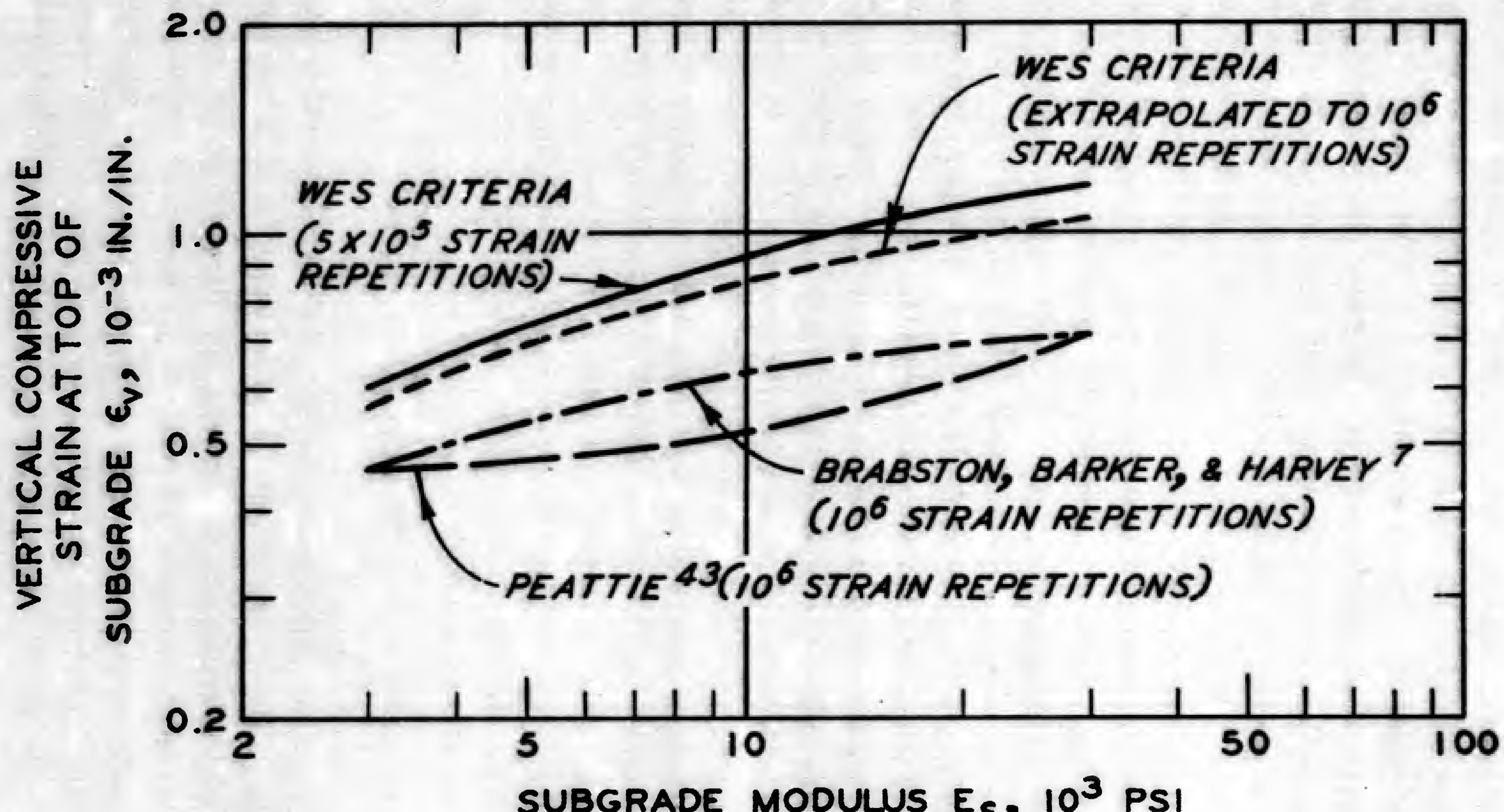


Figure 27. Relationship between subgrade strain and subgrade modulus

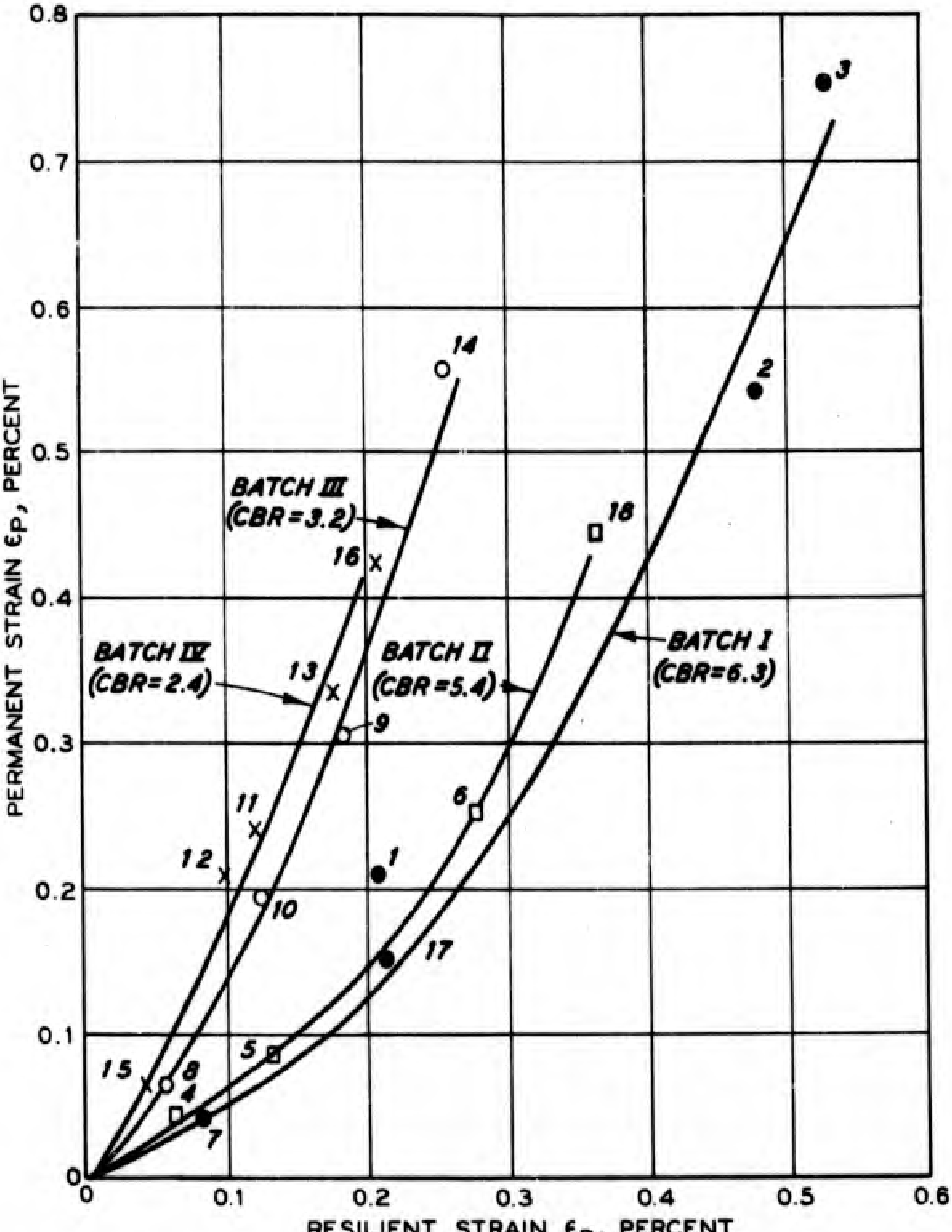


Figure 28. Preliminary results from laboratory tests to determine the effect of soil strength on the relationship between permanent and resilient strain

parameter to be considered in determining the fatigue strength.

Thus, an aircraft operating on a pavement would cause a certain amount of damage, the degree of which would depend on the temperature of the pavement. The damage then would have to be evaluated at all operational temperatures and would have to be accumulated in order to predict failure due to cracking. Therefore, the fatigue strength must be evaluated at a range of temperatures covering the operational temperatures. The test procedure recommended for determining fatigue life of bituminous concrete is presented in Appendix I. If it is not possible to conduct the tests on the specific mix to be used, then the relationship from Heukelom and Klomp[30] can be used. This relationship is also presented in Appendix I. Other methods or data may be available to a designer for a specific design situation which would provide the necessary criteria concerning the fatigue life of a bituminous mix.

HORIZONTAL TENSILE STRAIN CRITERIA FOR STABILIZED MATERIALS

As was the case for bituminous concrete, the method for developing the horizontal tensile strain criteria for stabilized base and subbase course materials was direct flexural testing of laboratory specimens. For bituminous-stabilized materials, the same methodology was used as for bituminous concrete, i.e., that which is given in Appendix I.

For cement- and lime-stabilized materials, the criteria were developed using the test procedures outlined in Appendix H. When, as with the bituminous-stabilized materials, flexural fatigue tests were not possible, then a preestablished relationship such as that shown in Figure 29 was used. Such data as presented in Figure 29 imply that the allowable strain is independent of the type or quality of the stabilized material. For this reason, when laboratory tests were conducted to determine the fatigue characteristics of a stabilized material, the differences in type and quality from laboratory material to field material were ignored.

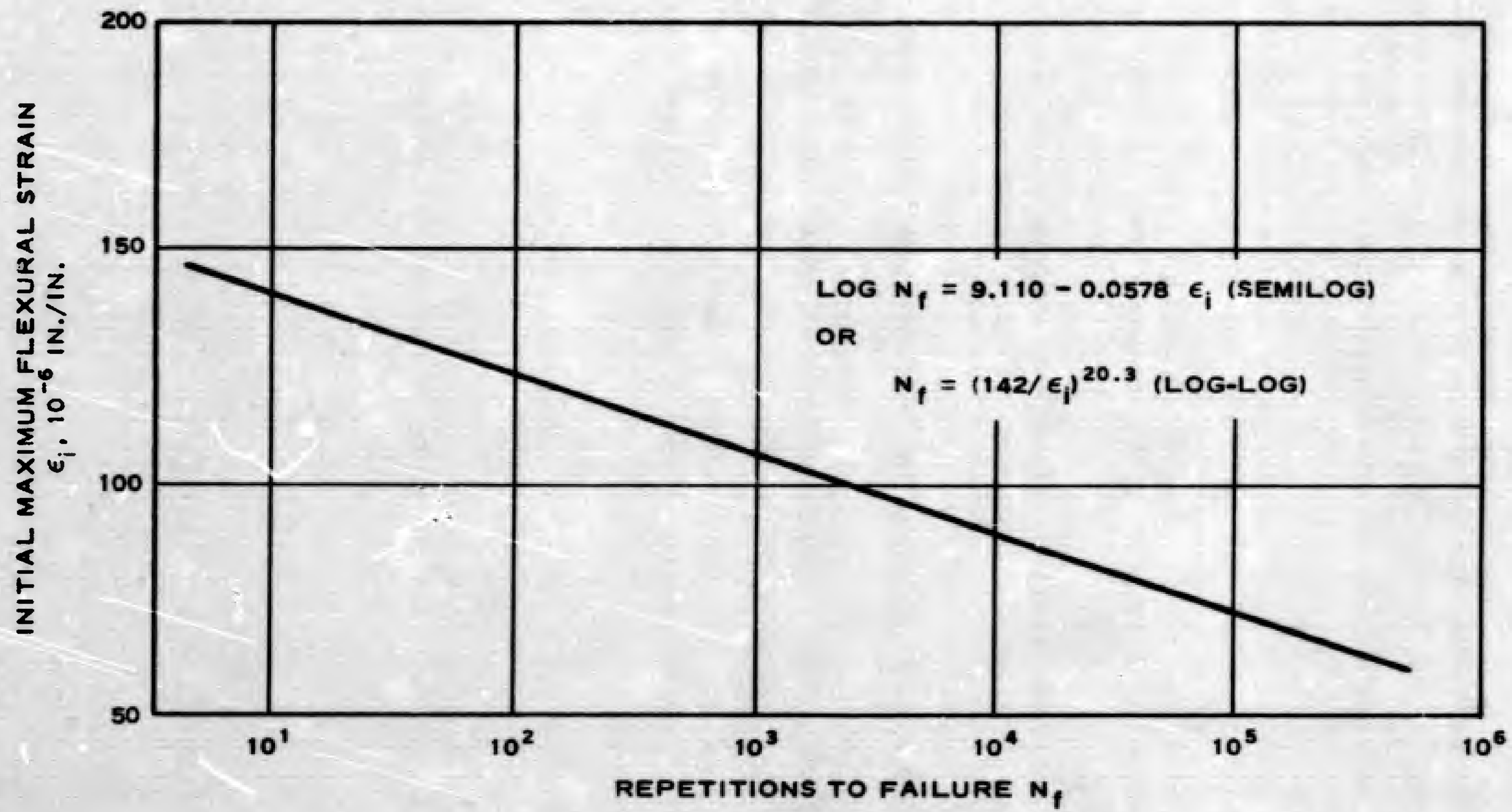


Figure 29. Fatigue life of flexural specimens (after Pretorius[6])

SUMMARY

The design procedure presented in this report provides the methodology for the design of three types of flexible pavement: conventional, bituminous concrete, and chemically stabilized. This capability is demonstrated by the design examples given in Appendix K. These three pavement types represent nearly all classes of flexible pavement being constructed at this time. The bases for design are the analytically determined strain values and experimental and laboratory determined material fatigue strengths. Thus, the procedure handles in a rational manner possible variations in the properties of different pavement materials. The adaptation of the cumulative damage concept permits the consideration of cyclic variation in bituminous materials due to the variation in temperature and the variation in subgrade strength resulting from freeze-thaw cycles. Although not considered in this report, the cumulative damage concept can be extended for consideration of traffic distribution with respect to aircraft wander, time, load, and aircraft type and speed. The extension of this capability is in progress. The first stage will include a more thorough consideration of traffic distribution with time and elementary treatment of the parameters of wander, load, and aircraft type.

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

The procedure presented in this report demonstrates that it is presently possible to handle in a rational manner a number of design parameters which are not now considered in the present CE and FAA design procedures. Since pavement research dollars have been invested in developing a theoretical approach to design, there is considerable advantage to adopting the procedure at an early date.

RECOMMENDATIONS

The following recommendations are offered:

a. Existing test data should be used for a more extensive verification of the design procedure. Along with the verification, a sensitivity study should be conducted to identify the most critical variables.

b. The design procedure should be put into use on an experimental basis. During this experimental use, emphasis should be placed on obtaining feedback for verification and/or modification of the procedure.

c. Work should continue on the extension of the procedure to more realistically consider the traffic variables. The variables presently identified are wander, load, aircraft type and speed, and time of operation.

d. The presently ongoing FAA state-of-the-art review should be used to begin research into environmental effects on pavements. Initial efforts in this area should be toward prediction of moisture conditions under pavement systems. Other areas of effort should be cold weather cracking, temperature effects on the modulus of bituminous concrete, and long-term deterioration of bituminous concrete surface courses.

APPENDIX A: LOCAL CLIMATOLOGICAL DATA ANNUAL SUMMARY FOR JACKSON, MISS.*

NARRATIVE CLIMATOLOGICAL SUMMARY

Jackson is about 45 miles east of the Mississippi River on the west bank of Pearl River about 150 miles north of the Gulf of Mexico. The nearby terrain is gently rolling with no local topographic features that appreciably influence the weather. The National Weather Service Office is nearly 7 miles east-northeast of the Jackson Post Office and over 5 miles southwest of the Ross Barnett Reservoir, which has approximately 50 square miles of water surface. Alluvial plains up to 3 miles wide extend along the river near Jackson where some levees have been built on both sides of the river. The largest floods produced crest stages of 37.2 feet (19.2 feet above flood stage) on December 21, 1961, and April 1, 1902.

Jackson's climate is significantly humid during most of the year, with a relatively short cold season and a rather long warm season. The proximity of the Gulf of Mexico and the prevalence of southerly winds amount to a maritime characteristic during the warm season that shifts the time of maximum daily mean temperature to near the end of July. In the cold season polar and arctic airmasses cover the area a significant portion of the time providing a continental modification of the climate to the extent of shifting the time of the minimum daily mean temperature to early January. Temperatures as high as 80° occasionally occur in midwinter and drop as low as 55° in midsummer. Subzero temperatures have been recorded twice in the 20th century (January 1940 and 1962).

Mean monthly precipitation ranges from about 4 inches to over 5 inches for the months of December through July while the relatively dry fall season provides significantly less precipitation with a minimum of a little over 2 inches for October. Although infrequent, tropical disturbances, including hurricanes and their remnants, that pass near or visit the Mississippi Coast in the summer and early fall, may bring several days of heavy rain. Occasionally during the summer the pressure distribution alters to bring westerly or northerly winds with hot, dry weather as the result. If these periods are prolonged, drought conditions may develop and the danger of fires increases. Snowfall averages less than 2 inches per season and the total is a trace or none in almost two-thirds of the seasons. Single storms frequently account for the significant portion of a season's snowfall. Severe ice storms, freezing rain and sleet with a destructive accretion of ice, occasionally cause major damage to wire lines and trees in the winter or early spring season.

Usually thunderstorms occur in each month, but at times one or more months in the October to March period have none. Generally the more intense rainfalls are associated with thunderstorms. The heaviest recorded rate of rainfall in the Jackson area was 0.77 inch in 5 minutes during a thunderstorm the night of March 3, 1964. Excessive rainfalls may occur in any season. In the late fall, winter, and early spring, thunderstorms may occur at any time of the night or day. They are usually associated with passing weather systems and are likely to be attended by higher winds than in the summer. In the winter about one-fifth of the days with rain have thunder; in the summer, nearly all. Thunderstorms are only occasionally accompanied by hail; most of that which falls is less than 5/8 inch in diameter. Hail of a damaging nature seldom occurs and usually then only in a small area.

Humidities of 90 percent or higher have occurred at any hour in the year. They are most frequent in the early morning hours. In the summer, at times there develops a combination of high temperatures together with high humidity; this usually builds up progressively for several days, and becomes oppressive for one or more days. Summer nights are frequently uncomfortable, partly because of the humid conditions, but more so because the wind becomes very light or calm in the late afternoon and at night. Relief is at times afforded by afternoon or evening thunderstorms that lower the temperature. Humidities of less than 50 percent occur on some days each month, usually in the early afternoon hours. Humidities drop under 30 percent on about one-quarter of the October and November days; the number of days with such low humidities diminishes in the other months. In July there may be none.

In the annual course of the normal mean daily temperatures, the greatest rise is early in April, and the greatest drop is in October. The average date for the last occurrence in the spring of temperatures as low as 32° is March 18 and the average date for the first such occurrence in the fall is November 8. Some low-lying or frost-susceptible places average later dates in the spring and earlier in the fall. On April 25, 1910, a temperature of 31° was recorded at Jackson, while on October 9, 1917, a temperature of 32° was noted. The mean freeze-free season is 235 days; in 1944 it lasted 287 days; in 1910, 187 days. The highest temperatures for the year range from the middle 90's to over 100° and the lowest temperature for the year is below 20° in about four-fifths of the years. The nights at times can remain uncomfortably warm. There have been occasions when the temperatures did not drop below 75° for 4 consecutive days. Minimum temperatures of 76° or higher have occurred between early June and late September; the lowest temperature, September 1, 1905, was 82°.

Over a year's time about half of the hourly winds range from 4 to 12 m.p.h. and nearly a third are 3 m.p.h. or less. For construction design purposes sustained winds around 70-75 m.p.h. have a 50-year mean recurrence interval 30 feet above ground. Each year there is some wind damage in the area mostly from the more severe gusts or sustained "straight-line" winds of severe local thunderstorms or windstorms. The most recent major tornado that damaged part of Jackson was in the late afternoon March 3, 1966, while the previous major one occurred in the early morning June 6, 1916.

---

* From "Local Climatological Data Annual Summary with Comparative Data, Jackson, Mississippi."[8]

METEOROLOGICAL DATA FOR THE CURRENT YEAR

Station: JACKSON, MISSISSIPPI   ALLEN C THOMPSON FIELD   Standard time used: CENTRAL   Latitude: 32° 19' N   Longitude: 90° 05' W   Elevation (ground): 310 feet   Year: 1973

Month	Temperature: Averages: Daily maximum	Daily minimum	Monthly	Extremes: Highest	Date	Lowest	Date	Degree days (Base 65°): Heating	Cooling	Precipitation: Total	Greatest in 24 hrs	Date	Snow, Ice pellets: Total	Greatest in 24 hrs	Date	Relative humidity: Hour 00	Hour 06	Hour 12	Hour 18 (Local time)
JAN	55.1	33.5	44.3	74	18+	13	13	634	1	4.59	1.12	7	T	T	9+	88	92	67	70
FEB	59.2	34.6	46.9	74	3	18	11	503	0	4.23	2.28	1	T	T	8	83	89	58	63
MAR	72.9	51.2	62.1	84	13	36	22	135	51	6.12	2.70	23-24	0.0	0.0		81	87	62	62
APR	73.1	51.8	62.5	83	21	31	11	146	77	9.44	4.98	15-16	0.0	0.0		76	81	54	56
MAY	82.7	59.1	70.9	94	10	45	18+	19	210	5.96	1.62	7	0.0	0.0		78	81	49	54
JUN	91.6	70.9	81.3	96	29	66	23+	0	498	0.32	0.20	5	0.0	0.0		79	83	51	55
JUL	93.7	73.5	83.7	97	16+	70	12	0	583	1.99	0.70	25-26	0.0	0.0		80	85	52	60
AUG	90.9	69.2	80.1	98	11	61	23	0	477	2.38	0.85	16-17	0.0	0.0		83	89	52	60
SEP	85.6	69.5	77.6	91	9	52	19	0	382	4.44	1.07	4-5	0.0	0.0		88	90	60	77
OCT	79.8	57.4	68.6	87	7+	37	30	61	181	2.72	0.98	15-16	0.0	0.0		90	93	56	75
NOV	73.1	50.3	61.7	84	26+	33	11	173	81	6.15	2.21	20-21	0.0	0.0		83	89	57	75
DEC	60.3	38.2	49.3	80	3	23	22+	486	5	6.71	3.88	24-25	0.3	0.3	20	74	79	53	65
YEAR	76.5	54.9	65.7	98	AUG. 11	13	JAN. 13	2157	2546	55.05	4.98	APR. 15-16	0.3	0.3	DEC. 20	82	87	57	64

Month	Wind &: Resultant: Direction	Resultant: Speed	Average speed	Fastest mile: Speed	Direction #	Date	Percent of possible sunshine	Average sky cover sunrise to sunset	Number of days: Sunrise to sunset: Clear	Partly cloudy	Cloudy	Precipitation .01 inch or more	Snow, Ice pellets 1.0 inch or more	Thunderstorms	Heavy fog	Temperatures: Maximum: 90° and above ‡	Maximum: 32° and below	Minimum: 32° and below	Minimum: 0° and below	Average daily solar radiation - langleys
JAN	03	1.0	8.8	34	SE	31	45	6.5	10	4	17	12	0	2	1	0	0	12	0	
FEB	35	1.8	8.0	33	SE	1	63	5.6	9	9	10	7	0	2	1	0	0	15	0	
MAR	17	4.2	9.3	34	SE	24	52	7.0	6	6	19	13	0	10	3	0	0	0	0	
APR	18	2.8	8.6	38	SE	18	49	6.0	8	8	14	10	0	5	1	0	0	1	0	
MAY	20	2.6	6.9	35	NW	23	57	4.7	13	9	9	11	0	9	1	3	0	0	0	
JUN	19	1.5	5.3	27	SW	13	51	7.2	2	12	16	5	0	8	3	24	0	0	0	
JUL	19	1.4	5.8	31	NE	2	51	6.1	4	16	11	12	0	11	1	30	0	0	0	
AUG	09	1.3	5.6	26	NW	14	58	4.5	14	13	4	8	0	13	4	22	0	0	0	
SEP	12	2.9	6.1	25	SE	4	34	6.7	5	8	17	13	0	4	3	4	0	0	0	
OCT	11	1.6	5.5	26	N	28	63	3.6	19	7	5	6	0	2	3	0	0	0	0	
NOV	16	4.2	8.4	42	NW	4	37	4.7	12	9	9	10	0	5	3	0	0	0	0	
DEC	18	1.4	10.1	33	SE	24	57	4.5	14	7	10	10	0	5	0	0	0	12	0	
YEAR	16	1.5	7.4	42	NW	NOV. 4	53	5.6	116	108	141	117	0	76	24	83	0	40	0	

NORMALS, MEANS, AND EXTREMES

Month	Temperature: Normal: Daily maximum	Daily minimum	Monthly	Extremes Ø: Record highest	Year	Record lowest	Year	Normal heating degree days (Base 65°)	Precipitation †: Normal total	Maximum monthly	Year	Minimum monthly	Year	Maximum in 24 hrs	Year
(a)	(b)	(b)	(b)	10		10		(b)	(b)	10		10		10	
J	56.4	35.8	47.1	82	1972	7	1966	569	4.53	8.23	1966	0.86	1969	1.86	1966
F	61.7	37.8	49.8	82	1972	11	1970	442	4.62	7.90	1965	2.32	1964	2.86	1965
M	68.7	43.4	56.1	86	1967	18	1968	313	5.63	10.92	1964	2.05	1966	3.37	1964
A	78.2	53.1	65.7	92	1970	30	1971	74	4.65	11.88	1964	1.25	1965	4.98	1973
M	85.0	60.4	72.7	99	1964	38	1971	6	4.38	10.82	1967	1.36	1965	3.27	1966
J	91.0	67.7	79.4	103	1969	49	1972+	0	3.40	3.78	1967	0.32	1973	1.84	1967
J	92.7	70.6	81.7	102	1970+	51	1967	0	4.27	6.28	1971	1.92	1965	1.67	1964
A	92.6	69.8	81.2	99	1972+	55	1966	0	3.59	6.65	1966	1.65	1963	3.45	1966
S	88.0	64.0	76.0	98	1972	35	1967	0	2.99	9.61	1965	0.36	1969	5.86	1965
O	80.1	51.5	65.8	91	1972+	30	1968	91	2.22	9.13	1970	0.00	1963	6.97	1964
N	68.5	42.0	55.3	88	1971	17	1966	301	3.87	6.44	1964	0.93	1967	4.21	1964
D	60.5	37.3	48.9	81	1971+	14	1966	504	5.04	9.82	1971	3.65	1965	4.51	1971
YR	77.1	52.8	65.0	103	JUN. 1969	7	JAN. 1966	2300	49.19	11.88	APR. 1964	0.00	OCT. 1963	6.97	OCT. 1964

Month	Snow, Ice pellets: Mean total	Maximum monthly	Year	Maximum in 24 hrs	Year	Relative humidity: Hour 00	Hour 06	Hour 12	Hour 18 (Local time)	Wind &: Mean speed	Prevailing direction	Fastest mile $: Speed #	Direction	Year
(a)	10	10		10		10	10	10	10	10		9	9	
J	0.2	1.5	1964	1.5	1964	84	87	64	71	9.2		38	N	1968
F	0.4	3.6	1968	3.6	1968	81	86	58	63	9.0		39	SE	1966
M	0.3	5.3	1968	5.3	1968	82	87	57	58	9.6		45	NW	1966
A	T	T	1971	T	1971	84	91	57	60	9.1		38	SE	1973
M	0.0	0.0		0.0		86	91	54	60	7.4		40	NW	1967
J	0.0	0.0		0.0		86	90	55	59	6.3		40	SE	1969
J	0.0	0.0		0.0		89	93	59	66	6.2		36	S	1970
A	0.0	0.0		0.0		89	93	60	69	5.8		40	NE	1969
S	0.0	0.0		0.0		89	93	59	72	6.6		43	SE	1965
O	0.0	0.0		0.0		89	93	54	72	6.7		36	N	1964
N	T	T	1971+	T	1971+	87	91	57	74	7.8		42	NW	1973
D	T	3.1	1963	1.8	1963	85	88	66	77	8.8		39	S	1966
YR	1.1	5.3	MAR. 1968	5.3	MAR. 1968	86	90	58	67	7.7		45	NW	MAR. 1966

Month	Pct. of possible sunshine	Mean sky cover sunrise to sunset	Mean number of days: Sunrise to sunset: Clear	Partly cloudy	Cloudy	Precipitation .01 inch or more	Snow, Ice pellets 1.0 inch or more	Thunderstorms	Heavy fog	Temperatures: Max.: 90° and above ‡	Max.: 32° and below	Min.: 32° and below	Min.: 0° and below	Average daily solar radiation - langleys
(a)	9	10	10	10	10	10	10	10	10	10	10	10	10	
J	47	6.8	8	5	18	11	*	2	3	0	*	14	0	
F	56	6.0	9	5	14	9	*	2	3	0	*	13	0	
M	61	6.1	9	8	14	10	*	6	2	0	0	6	0	
A	61	6.0	8	8	14	9	0	5	2	*	0	*	0	
M	65	5.5	10	10	11	9	0	6	1	5	0	0	0	
J	68	5.5	9	12	9	7	0	8	1	20	0	0	0	
J	63	6.1	5	15	11	11	0	11	1	24	0	0	0	
A	63	5.7	7	14	10	11	0	11	2	21	0	0	0	
S	59	5.6	9	10	11	9	0	5	2	12	0	0	0	
O	64	4.5	15	7	9	6	0	1	2	1	0	1	0	
N	53	5.4	10	8	12	8	0	2	3	0	0	5	0	
D	44	6.7	8	6	17	11	0	4	4	0	0	10	0	
YR	60	5.8	107	108	150	110	*	63	26	84	*	49	0	

Ø February 1964 through the current year. † Extremes are for period August 1963 through the current year. $ May 1964 through the current year. Means and extremes above are from existing and comparable exposures. Annual extremes have been exceeded at other sites in the locality as follows: Highest temperature 107 in July 1930; lowest temperature -5 in January 1940; maximum monthly precipitation 15.76 in November 1948; maximum precipitation in 24 hours 7.50 in December 1942; maximum monthly snowfall 10.6 in January 1940; maximum snowfall in 24 hours 10.6 in January 1940; Fastest mile wind 68 S in March 1952.

(a) Length of record, years, based on January data. Other months may be for more or fewer years if there have been breaks in the record.
(b) Climatological normals (1941-1970).
* Less than one half.
\+ Also on earlier dates, months, or years.
T Trace, an amount too small to measure.
\- Below zero temperatures are preceded by a minus sign.
‡ ≤ 70° at Alaskan stations.

The prevailing direction for wind in the Normals, Means, and Extremes table is from records through 1963.

Unless otherwise indicated, dimensional units used in this bulletin are: temperature in degrees F.; precipitation, including snowfall, in inches; wind movement in miles per hour; and relative humidity in percent. Heating degree day totals are the sums of negative departures of average daily temperatures from 65° F. Cooling degree day totals are the sums of positive departures of average daily temperatures from 65° F. Sleet was included in snowfall totals beginning with July 1948. The term "ice pellets" includes solid grains of ice (sleet) and particles consisting of snow pellets encased in a thin layer of ice. Heavy fog reduces visibility to 1/4 mile or less.

Sky cover is expressed in a range of 0 for no clouds or obscuring phenomena to 10 for complete sky cover. The number of clear days is based on average cloudiness 0-3, partly cloudy days 4-7, and cloudy days 8-10 tenths.

Solar radiation data are the averages of direct and diffuse radiation on a horizontal surface. The langley denotes one gram calorie per square centimeter.

& Figures instead of letters in a direction column indicate direction in tens of degrees from true North; i.e., 09 - East, 18 - South, 27 - West, 36 - North, and 00 - Calm. Resultant wind is the vector sum of wind directions and speeds divided by the number of observations. If figures appear in the direction column under "Fastest mile" the corresponding speeds are fastest observed 1-minute values.

\# To 8 compass points only.

AVERAGE TEMPERATURE

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1934	50.4	46.8	54.5	64.9	72.2	81.0	83.0	83.2	74.0	68.2	57.8	48.3	65.4
#1935	48.9	50.8	63.2	64.3	73.4	77.3	82.1	82.4	75.2	67.8	53.8	42.6	65.1
1936	46.2	47.0	62.0	61.8	72.4	80.2	81.6	82.4	80.3	65.8	52.6	50.4	65.2
1937	57.2	51.2	53.3	65.6	73.4	79.8	80.4	81.6	74.8	62.0	51.4	47.2	64.6
1938	49.7	56.8	66.1	63.6	72.6	77.5	81.2	82.1	75.8	66.0	55.3	48.0	66.2
#1939	52.0	52.4	61.0	62.9	72.5	80.8	84.1	81.9	80.2	67.2	52.4	51.2	66.5
1940	31.9	46.3	57.2	62.7	70.6	78.6	81.0	80.8	73.4	66.4	54.8	52.4	63.0
1941	49.6	45.4	50.8	66.9	73.8	80.2	83.2	82.1	79.0	72.9	52.5	51.2	65.6
1942	43.3	45.7	56.5	66.3	71.9	80.6	82.6	81.1	74.2	67.2	59.3	50.1	64.9
1943	48.6	52.1	54.6	66.6	76.8	83.2	83.2	85.6	74.1	68.2	52.4	48.4	65.7
1944	47.8	56.4	59.0	64.1	73.4	82.4	83.2	82.0	78.8	66.2	56.6	45.8	66.3
1945	46.4	52.8	64.2	66.4	69.6	79.4	80.8	81.0	78.2	64.4	58.8	43.7	65.5
1946	47.8	52.0	61.4	68.2	71.4	78.2	80.8	80.2	76.2	67.1	60.8	52.8	66.4
#1947	49.2	42.5	51.0	68.1	71.2	79.8	80.1	84.3	78.6	71.4	53.8	50.0	64.9
1948	39.6	51.3	60.0	69.0	73.8	82.0	84.5	81.4	73.8	64.4	57.4	52.4	65.8
1949	53.8	54.4	57.2	64.0	75.1	80.2	82.4	81.4	75.6	70.1	55.4	52.2	66.8
1950	59.8	55.6	55.0	61.5	75.9	79.7	79.9	80.0	75.1	68.8	52.3	44.4	65.7
1951	49.1	51.8	58.8	62.8	73.9	81.0	83.7	85.7	78.3	68.7	51.3	52.8	66.5
1952	55.1	54.5	55.5	62.0	72.8	84.6	84.9	84.0	74.8	58.9	53.6	48.5	65.8
1953	52.1	50.8	62.5	62.8	73.4	84.6	82.4	81.6	77.1	68.0	54.5	45.9	66.5
1954	49.6	54.9	56.1	70.4	67.8	82.1	84.5	86.0	79.4	67.5	54.0	48.5	66.7
#1955	46.1	49.9	60.0	68.6	76.1	75.5	80.5	80.2	78.4	63.7	53.4	48.5	65.1
1956	44.4	54.6	55.3	62.4	74.7	76.6	81.9	80.8	74.1	66.7	53.1	55.9	65.1
1957	49.5	57.6	54.3	65.8	73.0	79.4	81.5	80.2	73.3	61.3	55.6	50.3	65.1
1958	42.0	40.4	51.0	64.6	72.1	78.9	81.3	79.7	77.3	63.8	56.8	44.3	62.7
1959	43.5	50.9	54.6	63.0	74.9	78.1	80.8	81.4	76.4	68.3	51.6	48.7	64.4
1960	45.5	43.5	47.5	66.1	69.9	80.0	83.9	80.5	76.2	66.7	55.8	44.5	63.3
1961	40.6	52.9	60.5	61.0	70.4	75.5	79.1	79.1	75.9	64.1	55.5	48.2	68.6
1962	42.7	57.2	52.5	62.6	75.3	78.5	84.3	83.2	77.2	68.3	53.6	46.5	65.1
#1963	40.5	43.5	61.5	68.1	74.4	80.7	81.1	81.9	75.0	70.0	56.4	38.6	64.3
#1964	45.3	44.4	56.3	68.1	74.7	80.7	81.3	81.7	77.1	62.5	59.4	50.5	65.2
1965	48.5	47.2	50.2	67.8	74.0	77.7	81.5	79.5	76.3	62.6	60.1	49.4	64.6
1966	40.8	46.3	54.7	64.8	70.9	76.1	82.4	78.3	74.4	61.1	56.2	46.5	62.7
1967	46.4	44.2	60.5	69.3	69.6	78.3	77.3	77.0	69.8	61.4	52.7	50.1	63.1
1968	43.4	39.9	53.9	65.9	71.2	79.6	80.7	80.9	73.3	66.2	52.7	45.7	62.8
1969	48.0	48.1	49.3	65.1	72.3	80.3	83.5	79.6	75.2	66.2	52.4	47.1	63.9
1970	41.2	46.5	54.0	68.1	72.9	79.1	80.8	81.6	80.1	64.7	52.8	52.7	64.5
1971	48.7	49.1	52.4	62.2	67.9	79.9	81.0	80.5	78.3	70.1	54.7	57.9	65.2
1972	51.5	51.2	58.8	66.4	71.9	79.6	80.4	82.8	81.5	68.0	52.0	50.1	66.2
1973	44.3	46.9	62.1	62.5	70.9	81.3	83.7	80.1	77.6	68.6	61.7	49.3	65.7
RECORD MEAN	48.2	50.9	57.3	65.4	72.5	79.7	81.9	81.5	77.1	66.6	55.7	49.5	65.5
MAX	58.6	61.8	68.8	77.0	84.1	91.0	92.6	92.5	88.8	79.9	68.2	60.0	76.9
MIN	37.7	39.9	45.8	53.7	60.9	68.4	71.1	70.4	65.3	53.2	43.1	38.9	54.0

HEATING DEGREE DAYS

JACKSON, MISSISSIPPI

Season	July	Aug.	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	Total
#1934-35	0	0	5	27	235	523	516	397	147	105	4	0	1959
1935-36	0	0	4	35	371	693	579	536	156	150	0	0	2524
1936-37	0	0	0	77	388	459	267	399	368	129	2	0	2089
1937-38	0	0	7	166	411	554	488	251	85	112	8	0	2082
#1938-39	0	0	9	91	321	527	411	357	166	125	6	0	2013
1939-40	0	0	0	105	375	431	1025	541	258	141	13	0	2889
1940-41	0	0	29	49	316	399	479	549	437	23	0	0	2281
1941-42	0	0	0	22	375	429	675	538	293	60	3	0	2395
1942-43	0	0	28	53	216	471	524	364	333	73	0	0	2062
1943-44	0	0	4	121	393	529	541	286	214	108	17	0	2213
1944-45	0	0	0	75	266	594	581	354	122	62	40	0	2094
1945-46	0	0	0	97	235	657	539	367	157	30	5	1	2088
1946-47	0	0	0	44	181	386	506	630	434	48	5	0	2234
#1947-48	0	0	3	5	339	468	789	413	234	42	3	0	2296
1948-49	0	0	12	82	249	405	372	315	265	105	0	0	1805
1949-50	0	0	10	36	298	410	215	271	334	155	0	0	1729
1950-51	0	0	0	17	393	636	486	391	242	142	3	0	2310
1951-52	0	0	0	57	426	394	331	307	298	132	8	0	1953
1952-53	0	0	0	217	350	503	394	391	126	112	3	0	2096
1953-54	0	0	0	74	315	583	479	286	311	33	70	0	2151
#1954-55	0	0	0	116	321	505	577	425	242	45	0	0	2231
1955-56	0	0	0	135	385	517	635	314	310	133	0	3	2432
1956-57	0	0	0	30	377	298	481	241	328	114	24	0	1893
1957-58	0	0	17	148	301	449	707	682	429	89	16	0	2838
1958-59	0	0	0	83	276	632	663	395	321	127	3	0	2500
1959-60	0	0	0	71	412	498	606	620	550	57	49	0	2863
1960-61	0	0	0	71	276	626	748	336	169	172	37	2	2437
1961-62	0	0	3	109	308	516	690	236	387	151	2	0	2404
#1962-63	0	0	0	73	340	567	752	597	179	45	8	0	2561
#1963-64	0	0	3	27	265	810	602	592	280	44	0	0	2623
1964-65	0	0	0	124	219	465	512	500	459	48	2	0	2329
1965-66	0	0	9	129	173	481	747	520	326	98	13	0	2496
1966-67	0	0	0	155	270	581	570	574	192	20	26	0	2388
1967-68	0	0	35	141	368	461	662	721	362	64	12	0	2826
1968-69	0	0	0	93	369	592	523	470	476	65	3	0	2591
1969-70	0	0	0	95	372	550	741	516	340	72	23	0	2709
1970-71	0	0	0	85	367	388	507	444	394	147	40	0	2372
1971-72	0	0	0	19	328	235	431	410	216	76	1	0	1716
1972-73	0	0	4	71	400	466	634	503	135	146	19	0	2378
1973-74	0	0	0	61	173	486							

TOTAL PRECIPITATION

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1934	1.24	5.09	5.08	1.93	3.33	8.19	1.45	2.84	3.40	1.70	7.45	6.16	47.86
#1935	4.19	1.90	8.28	5.42	6.14	7.60	1.80	4.23	1.78	1.49	2.67	4.38	49.88
1936	5.17	6.66	3.08	5.27	2.52	3.37	3.64	3.68	0.95	0.39	3.00	8.04	46.37
1937	11.48	4.35	2.92	4.43	2.61	3.90	2.46	5.92	2.07	3.53	3.05	3.70	50.42
1938	5.54	3.86	6.13	10.13	1.71	5.11	7.82	3.36	0.52	0.98	2.36	3.30	50.82
#1939	5.35	9.48	5.39	1.76	3.95	4.75	1.81	2.45	1.60	0.69	0.85	5.26	43.34
1940	3.04	4.91	3.67	7.29	1.37	4.09	10.68	1.83	1.08	1.17	5.72	9.46	54.31
1941	2.42	2.80	6.85	4.05	2.03	2.73	3.45	3.38	2.31	5.35	2.20	5.52	43.09
1942	2.29	2.84	4.91	0.80	2.22	2.31	4.34	11.39	3.73	3.32	0.86	10.24	49.25
1943	1.31	3.73	6.42	5.40	1.55	5.01	3.56	0.68	2.45	0.82	2.47	3.80	37.20
1944	3.77	6.61	9.97	7.73	6.02	0.52	1.58	4.65	1.03	0.28	5.62	6.08	53.86
1945	2.50	8.35	9.87	3.15	3.76	9.27	5.91	4.02	1.68	4.79	1.99	3.94	59.21
1946	7.88	8.40	7.57	1.17	10.19	4.11	7.63	0.23	0.79	1.18	8.40	3.62	61.17
#1947	12.01	1.15	4.71	9.73	4.27	2.54	0.98	2.37	3.16	1.04	7.81	4.42	54.19
1948	3.28	5.08	5.37	4.13	4.46	1.42	1.54	6.25	3.87	0.80	15.76	3.77	56.33
1949	8.13	5.25	11.26	2.81	4.16	5.46	6.24	1.14	5.37	3.52	0.01	2.55	55.90
1950	8.26	9.28	5.46	2.98	4.12	3.47	8.12	3.18	1.53	1.30	2.94	3.82	54.46
1951	8.63	3.39	5.54	5.46	0.16	2.63	3.30	2.19	3.08	0.33	1.59	5.95	42.25
1952	2.08	2.74	3.58	2.21	6.22	1.59	1.46	1.51	2.10	T	2.61	6.06	31.66
1953	4.61	7.85	4.88	8.44	7.12	1.36	3.40	2.27	0.26	0.77	0.84	4.55	46.33
1954	4.87	1.48	2.91	2.68	8.30	2.10	2.65	2.17	2.98	1.48	2.47	3.61	37.70
#1955	3.91	5.43	2.14	7.16	5.18	0.92	13.13	0.52	0.22	0.59	3.37	2.90	45.41
1956	2.13	7.44	6.52	4.67	3.58	7.37	1.71	3.27	0.04	3.46	1.55	5.03	46.77
1957	3.85	2.60	6.14	4.75	3.70	8.92	3.22	0.93	8.29	2.79	9.78	3.09	58.00
1958	3.92	3.61	5.19	6.83	4.78	8.77	5.33	2.87	8.30	0.95	2.40	2.27	55.22
1959	3.73	4.09	4.88	3.86	4.69	2.16	8.81	2.76	3.03	4.18	2.33	5.96	50.46
1960	5.29	4.36	5.22	1.48	3.95	2.18	1.09	10.23	2.10	2.39	2.45	4.21	45.13
1961	3.07	6.59	8.90	2.00	2.96	7.49	8.55	3.96	1.22	0.84	8.33	11.16	65.07
1962	7.06	2.75	3.61	8.55	2.59	1.36	1.49	2.91	2.34	1.70	2.88	3.25	40.49
#1963	5.33	2.82	1.93	2.44	1.47	3.29	6.28	1.65	0.88	0.00	4.53	4.39	35.03
1964	5.21	2.32	10.92	11.88	2.18	2.45	5.17	6.43	3.67	7.83	6.44	6.95	71.45
1965	2.86	7.90	6.66	1.25	1.36	2.54	1.92	3.61	9.61	0.84	1.79	3.65	43.99
1966	8.23	7.84	2.05	5.76	7.96	1.45	3.14	6.65	4.87	1.63	3.66	4.83	58.07
1967	1.86	3.56	2.32	1.66	10.82	3.76	4.05	3.49	1.80	2.78	0.93	8.66	45.71
1968	4.56	2.54	2.83	7.20	7.54	1.02	2.24	3.90	1.95	0.28	5.55	5.58	45.19
1969	0.86	3.02	4.90	6.59	1.52	1.29	5.40	3.37	0.56	2.26	1.87	7.22	38.90
1970	2.09	2.63	5.40	2.75	2.43	2.54	2.34	5.64	6.47	9.13	2.70	4.23	48.36
1971	3.02	5.68	7.68	6.86	8.05	3.40	6.28	2.64	6.[illegible]	0.09	2.54	9.82	62.06
1972	5.94	3.09	5.57	2.44	4.52	2.01	3.31	2.84	5.04	2.08	3.52	9.67	50.03
1973	4.59	4.23	6.12	9.44	5.96	0.32	1.99	2.34	4.44	2.72	6.15	6.71	55.05
RECORD MEAN	4.76	4.76	5.48	5.20	4.43	3.79	4.40	3.54	2.75	2.19	3.66	5.57	50.53

TOTAL SNOWFALL

Season	July	Aug.	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	Total
#1934-35	0.0	0.0	0.0	0.0	0.0	0.0	3.6	0.0	0.0	0.0	0.0	0.0	3.6
1935-36	0.0	0.0	0.0	0.0	0.0	2.5	2.6	0.0	0.0	0.0	0.0	0.0	5.1
1936-37	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1937-38	0.0	0.0	0.0	0.0	0.0	T	0.0	0.0	0.0	0.0	0.0	0.0	T
1938-39	0.0	0.0	0.0	0.0	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2
1939-40	0.0	0.0	0.0	0.0	0.0	0.0	10.6	1.0	0.0	0.0	0.0	0.0	11.6
1940-41	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	T	0.0	0.0	0.0	T
1941-42	0.0	0.0	0.0	0.0	0.0	0.0	T	T	T	0.0	0.0	0.0	T
1942-43	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	T	0.0	0.0	0.0	T
1943-44	0.0	0.0	0.0	0.0	0.0	T	1.0	0.0	0.0	0.0	0.0	0.0	1.0
1944-45	0.0	0.0	0.0	0.0	0.0	T	T	0.0	0.0	0.0	0.0	0.0	T
1945-46	0.0	0.0	0.0	0.0	0.0	T	T	0.0	0.0	0.0	0.0	0.0	T
1946-47	0.0	0.0	0.0	0.0	0.0	T	0.0	T	T	0.0	0.0	0.0	T
#1947-48	0.0	0.0	0.0	0.0	0.0	T	5.2	T	0.0	0.0	0.0	0.0	5.2
1948-49	0.0	0.0	0.0	0.0	0.0	0.0	4.0	0.0	T	0.0	0.0	0.0	4.0
1949-50	0.0	0.0	0.0	0.0	0.0	T	0.0	0.0	0.0	0.0	0.0	0.0	T
1950-51	0.0	0.0	0.0	0.0	T	T	2.4	1.8	T	0.0	0.0	0.0	4.2
1951-52	0.0	0.0	0.0	0.0	T	0.0	0.0	0.0	0.0	0.0	0.0	0.0	T
1952-53	0.0	0.0	0.0	0.0	T	0.0	T	0.0	0.0	0.0	0.0	0.0	T
#1953-54	0.0	0.0	0.0	0.0	0.0	T	T	0.0	T	0.0	0.0	0.0	T
1954-55	0.0	0.0	0.0	0.0	0.0	T	0.0	0.0	0.0	0.0	0.0	0.0	T
1955-56	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1956-57	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	T	0.0	0.0	0.0	T
1957-58	0.0	0.0	0.0	0.0	T	0.0	T	2.5	0.0	0.0	0.0	0.0	2.5
1958-59	0.0	0.0	0.0	0.0	0.0	T	T	T	0.0	0.0	0.0	0.0	T
1959-60	0.0	0.0	0.0	0.0	0.0	T	T	9.1	T	0.0	0.0	0.0	9.1
1960-61	0.0	0.0	0.0	0.0	0.0	T	T	T	0.0	0.0	0.0	0.0	T
1961-62	0.0	0.0	0.0	0.0	0.0	0.0	4.0	0.0	0.0	0.0	0.0	0.0	4.0
#1962-63	0.0	0.0	0.0	0.0	0.0	0.0	T	1.0	0.0	0.0	0.0	0.0	1.0
1963-64	0.0	0.0	0.0	0.0	0.0	3.1	1.5	T	T	0.0	0.0	0.0	4.6
1964-65	0.0	0.0	0.0	0.0	T	T	T	T	T	0.0	0.0	0.0	T
1965-66	0.0	0.0	0.0	0.0	0.0	0.0	T	0.0	0.0	0.0	0.0	0.0	T
1966-67	0.0	0.0	0.0	0.0	T	T	0.0	T	0.0	0.0	0.0	0.0	T
1967-68	0.0	0.0	0.0	0.0	0.0	T	T	3.6	5.3	0.0	0.0	0.0	8.9
1968-69	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1969-70	0.0	0.0	0.0	0.0	0.0	T	T	0.0	T	0.0	0.0	0.0	T
1970-71	0.0	0.0	0.0	0.0	0.0	0.0	T	T	T	T	0.0	0.0	T
1971-72	0.0	0.0	0.0	0.0	T	0.0	0.0	T	0.0	0.0	0.0	0.0	T
1972-73	0.0	0.0	0.0	0.0	0.0	0.0	T	T	0.0	0.0	0.0	0.0	T
1973-74	0.0	0.0	0.0	0.0	0.0	0.3							

Record mean values above (not adjusted for instrument location changes listed in the Station Location table) are means for the period beginning in 1909.

# indicates a break in the data sequence during the year, or season, due to a station move or relocation of instruments. See Station Location table.

03940

STATION LOCATION

Location	Occupied from	Occupied to	Airline distance and direction from previous location	Latitude North	Longitude West	Elevation above Sea level: Ground at temperature site	Elevation above Ground: Wind instruments	Extreme thermometers	Psychrometer	Telepsychrometer	Tipping bucket rain gage	Weighing rain gage	8" rain gage	Hygrothermometer	Elevation above Sea level: Pyranometer	Remarks
COOPERATIVE																
A&V Depot, 430 S. State Street	6-1893	7-1899		32° 18'	90° 11'	a294		b					b			a - approximate value. b - ground exposure.
Western Union Office 530 E. Capitol Street	8-1899	7-1906	1000' NW	32° 18'	90° 11'	a294		b					b			
136 Adams Street	8-1906	2/21/08	1.25 mi. W	32° 18'	90° 12'	a294		b					b			
*Board of Trade Building 210 E. Capitol Street	2/21/08	3/19/10	1 mile E	32° 18'	90° 11'	a294		5					3			*Office of 2nd Order Station appears to have moved, but thermometers and rain gage were apparently in SITU on Post Office lawn February 1908 - December 1929.
*Board of Trade Building 230-1/2 E. Capitol Street	3/19/10	12/31/20	No change	32° 18'	90° 11'	a294		5					3			
*Post Office Building 245 E. Capitol Street	1/01/21	11/30/29	No change	32° 18'	90° 11'	a294		5					3			
SW corner S. State Street and Silas Brown Street	12-1929	6-1931	1 mile S	32° 17'	90° 11'	a294		5					3			
Jackson Municipal Airport	7-1931	6-1935	4 miles NW	32° 20'	90° 13'	315		5	5				3			
701 E. Silas Brown St.	7-1935	5-1939	4 miles SE	32° 17'	90° 11'	a204		5					3			
AIRPORT																
Administration Building Jackson Municipal Airport	6-1939	6/05/55	4 miles NW	32° 20'	90° 13'	315	c46	d21	d21		e20	e21	e20			c - approximate, 60 feet to 10/12/47. d - 5 feet to 10/12/47. e - 4 feet to 10/12/47.
Operations Building Jackson Municipal Airport Hawkins Field	6/06/55	7/08/63	200' WSW	32° 20'	90° 13'	305	#30	6	5		4	4	4			# - Moved 145' SE atop creosoted pole 2/17/59.
Terminal Building Allen C. Thompson Field Jackson Municipal Airport	7/08/63	To date	8.5 mi. E	32° 19'	90° 05'	g310	20	i47	i47		j4 m44	h5 k4	h4 k3	f5		f - Commissioned 2400 feet W of office 2/11/64. g - 330 feet to 2/11/64. h - 44 feet to 2/5/64. i - Decommissioned 3/10/64. j - Added 1/1/65. k - Elevation effective 9/13/67. m - Effective 8/11/73.

Requests for additional climatic information should be addressed to: Director, National Climatic Center, Federal Building, Asheville, N. C. 28801

Sale Price: 15 cents per copy. Checks and money orders should be made payable to Department of Commerce, NOAA. Remittances and correspondence regarding this publication should be sent to: National Climatic Center, Federal Building, Asheville, N. C. 28801. Attn: Publications.

USCOMM-NOAA-ASHEVILLE - 1000

APPENDIX B: LABORATORY PROCEDURES FOR DETERMINING THE RESILIENT PROPERTIES OF SUBGRADE SOILS AFFECTED BY FROST MELTING

1. The objective of this test procedure is to determine resilient moduli of airport subgrade materials during thaw conditions using resilient triaxial techniques. The test is similar to a standard triaxial compression test, the primary exceptions being that the deviator stress is applied repetitively and at several stress levels and that, if the soil is frost-susceptible, the test is performed on specimens that have been previously frozen and thawed. Use of this procedure allows testing of soil specimens in a repetitive stress state in an attempt to simulate conditions that occur during the period of thaw weakening of a soil under a pavement subjected to moving wheel loads.

2. Test procedures are presented that are applicable to each of the following three types of soil, which exhibit different behavior under frost action and/or repeated loading:

- a. Type A: Cohesive soils such as clay, or soils whose behavior is significantly influenced by their clay fraction.
- b. Type B: Cohesionless soils, such as silt, silty sand, or silty gravel, which exhibit moisture migration and ice segregation upon freezing and which exhibit a strong dependence of resilient modulus upon minor principal stress.
- c. Type C: Cohesionless soils, such as clean sand and gravel, which develop little or no ice segregation upon freezing but whose resilient modulus is strongly dependent upon minor principal stress.

In certain cases, it will be necessary to subject specimens to freezing tests to determine whether the material is of Type B or Type C. The freezing test described by Kaplar[44] is recommended for this purpose. Type C soils are defined for this test as those samples in which the average rate of heave is less than 0.5 mm/day.

3. The procedures presented herein include the following basic conditions:

- a. Types A and B materials should be tested in the thawed condition following open-system freezing of saturated specimens. No drainage should be permitted during thawing.

b. Type C material should be tested in the fully saturated state without prior freezing and thawing.

c. In Series I tests, no drainage should be permitted under the applied all-around confining pressure in materials of Types A, B, and C. Also, no drainage should be permitted during repetitions of deviator stress.

d. In Series II tests on materials of Types A and B, specimens should be drained and consolidated after thawing, and specimens of Type C material should be drained and consolidated after saturation under expected overburden pressure only. No further drainage should be permitted under incremental all-around confining pressure or under repetitions of deviator stress.

e. The highest resilient strain recorded between the tenth and two hundredth repetitions from Series I tests should be used for computation of a resilient modulus that will be applicable from the onset of the thaw period until all frost has left the pavement substructure.

f. The lowest resilient strain recorded between the first and two hundredth repetitions from Series II tests should be used for computation of a resilient modulus that will be applicable from the end of the thaw period until the time at which 80 percent recovery is estimated to have occurred.

DEFINITIONS

4. Symbols and terms used in this procedure are defined as follows:

a. σ_1 = total axial stress (major principal stress) in the triaxial test.

b. σ_3 = total radial stress and all-around confining pressure (minor principal stress) in the triaxial test.

c. $\sigma_d = \sigma_1 - \sigma_3$ = repeated deviator stress.

d. ε_1 = total axial strain caused by σ_d.

e. ε_{R_1} = resilient axial strain caused by σ_d at a particular number of stress repetitions.

f. $M_R = \dfrac{\sigma_d}{\varepsilon_{R_1}}$ = resilient modulus.

g. $\theta = \sigma_1 + 2\sigma_3 = \sigma_d + 3\sigma_3$ = sum of the principal stresses in the triaxial state of stress.

h. σ_1/σ_3 = principal stress ratio.

i. Load duration: The time interval over which the specimen is subjected to a deviator stress.

SPECIMEN PREPARATION

CLEAN SANDS AND GRAVELS

5. Soil specimens used in this test are generally similar to those used in the standard triaxial compression test, except that if the soil contains more than about 10 percent of particles retained on a No. 10 sieve the specimen diameter should be at least 2.5 to 3.0 in. The specimen height should be at least twice the diameter. Methods for laboratory preparation of remolded specimens and for back-pressure saturation, if required, are indicated in Engineering Manual EM 1110-2-1906, "Laboratory Soils Testing."[45]

COHESIVE FINE-GRAINED SOILS, SILTS, AND SILTY SANDS

6. Undisturbed or laboratory molded specimens may be used. Because the specimen should be tapered for freeze-thaw testing to minimize restraint to heave, laboratory compacted specimens prepared in tapered molds are preferable. Use of tapered specimens requires trimming to cylindrical shape prior to triaxial testing, however, and it has been found convenient to freeze tapered specimens of large enough diameter to produce at least four 1.4-in. triaxial specimens. In fine-grained soils, the latter are produced by core-drilling the large frozen specimens. Tapered specimens approximately 5.5 in. in diameter and 6 in. high have been used for this purpose. The procedures to be employed are similar to those described by Kaplar.[44] These procedures are:

a. Screen all material through a No. 4 mesh sieve to remove all larger-than-sand particles and to break up clods of soil.

b. Thoroughly mix a batch of 10 to 14 lb (depending on the required density).

c. Wet the soil to the desired water content, thoroughly mix, seal in a closed container, and allow to stabilize

24 hr prior to molding.

d. Compact to the required compaction effort in a tapered (1°) 6-in.-high steel molding cylinder with a removable base (inside diameter 5.70 at top and 5.45 in. at bottom). Experience has shown that only a slightly higher density (1 pcf) will be obtained in the tapered mold than in a 6-in. mold for the same number of blows.

e. Eject the molded specimen from the tapered steel mold and transfer the compacted specimen to a tapered Plexiglas cylinder (see Figure B1).

f. Weigh and calculate density.

g. Attach a saturation base containing a saturated porous stone and filter paper.

h. Weigh.

SILTY OR CLAYEY GRAVELS

7. Silty or clayey gravels cannot be prepared in the same manner as the finer grained soils because the former cannot be readily cored when frozen, as outlined in Paragraph 6 for other cohesive or silty soils. For this reason, soils containing gravel must be molded to the diameter required (approximately four times maximum aggregate size) for the triaxial test even though side restraint to heaving, accomplished by the Teflon linings, is greater in specimens frozen in straight-walled cylinders. Segmented or split-ring cylinders have been used with success for freezing tests, but the frozen specimen is often bent because of nonuniform heaving. Additional problems develop from finer grained material filling the gaps between split rings and the resulting irregular surface and restraint to consolidation during thawing. Given the present state-of-the-art, it is suggested that silty or clayey gravels be compacted in the manner prescribed above for clean sands and gravels. A right cylinder split along its length has been found at the U. S. Army Cold Regions Research and Engineering Laboratory (CRREL) to provide ease of ejection. The cylinder must be restrained radially during compaction and freezing.

SPECIMEN FREEZING

8. CRREL uses freezing cabinets in which four 5.5-in.-diam specimens may be frozen simultaneously. The cabinets are designed to

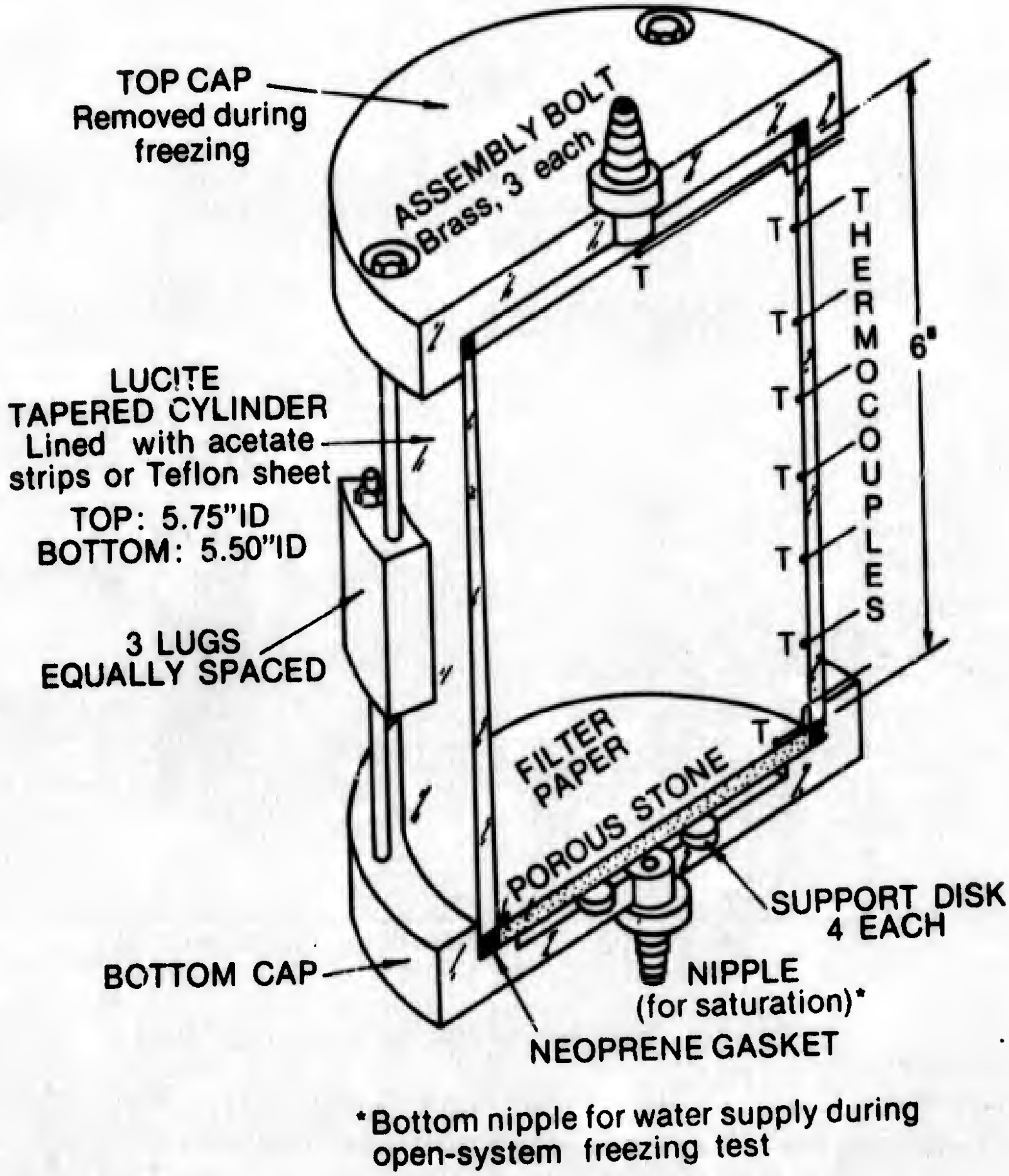


Figure B1. Inside-tapered freezing cell

operate at temperatures as low as -20° F in a room of 38 to 40° F ambient temperature. Details of a typical soil freezing cabinet are shown in Figure B2. Each cabinet is cooled by a 1/4-hp refrigeration unit. A thermoregulator is used to control the temperature and a fan is used to obtain temperature uniformity within the cabinet. The bottom of each cabinet is open to the ambient room temperature. Water is supplied to the baseplates of each specimen from an adjustable water level device. Granulated cork is used as insulation around each

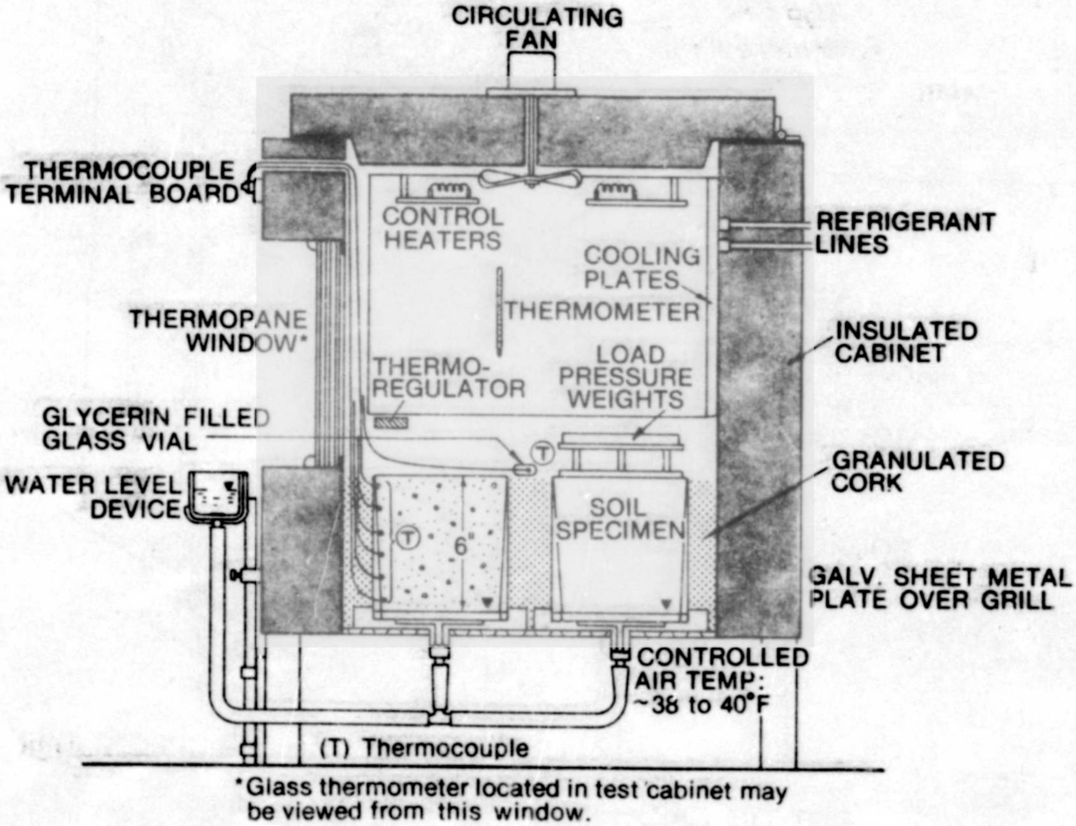


Figure B2. CRREL freezing cabinet (from Kaplar[44])

specimen. This material is preferable to polystyrene foam beads because of the clinging nature of statically charged foam beads.

9. A thermoelectric cooling unit (Peltier battery) has been used at the University of New Hampshire as an alternative to freezing cabinets. This unit, which provides unidirectional freezing and thawing, is described by Leary et al.[46] Figure B3 shows a cross section of the unit. (The Lucite rings shown in the figure should be replaced with the tapered Plexiglas mold as previously described.) All of the freezing cylinder except for the upper surface is fully encased in rigid foam insulation. The Peltier battery is placed on a cold plate in direct contact with the upper surface of the specimen. The thermoelectric

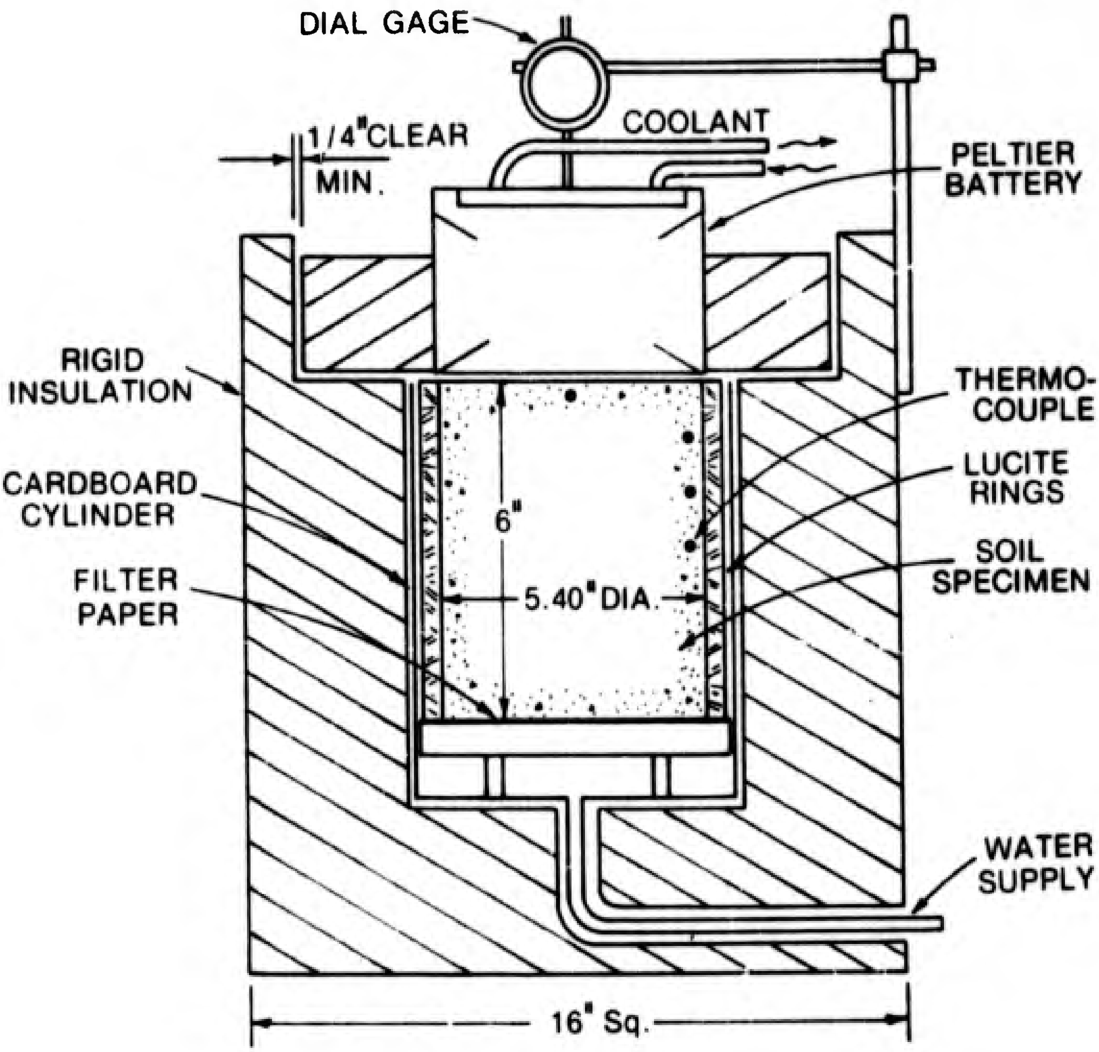


Figure B3. University of New Hampshire freezing test equipment using thermoelectric cooling unit (after Zoller[47])

cooling unit must have a rated maximum heat-pumping capacity of 600 BTU/hr at 4.5 A and 12 v DC. The actual capacity is a function of the temperature difference between the hot and cold sides of the Peltier battery. For a typical installation where the ambient temperature is 68° F and the cold plate temperature is 23° F, the actual heat-pumping capacity will be 136 BTU/hr.

10. The procedures for specimen freezing outlined in the following paragraphs are applicable to the use of freezing cabinets of the

type employed at CRREL. With only slight adaptations, however, the techniques can be used for thermoelectric cooling units.

CLEAN SANDS AND GRAVELS

11. Freezing of these materials does not induce moisture migration, and it is believed that tests on saturated specimens, without freezing, will yield results that will be adequate for the thaw condition. It is therefore suggested that clean sands and gravels be molded and prepared for testing as outlined in Paragraphs 6 and 23, insuring that saturation is obtained.

COHESIVE FINE-GRAINED SOILS,
SILTS, AND SILTY SANDS

12. This procedure provides a severe condition of frost action, with three freeze-thaw cycles on a saturated specimen with water freely available.

a. Place the compacted specimen contained in the tapered Plexiglas mold (approximately 5.5 in. diameter) in a +40° F environment.

b. Place a filter paper and porous stone on the upper surface and apply a 20-lb load to minimize swelling.

c. Connect the baseplate to a degassed water supply with the water level held 1 in. above the bottom of the specimen. Raise the water level 1 in. per hour until it is even with the top of the specimen. Then raise the level in 3-in. increments every 2 to 3 hr until the water level is 12 in. above the top of the specimen. Maintain this condition for 48 hr or until free water is visible over the top of the specimen. Weigh and calculate the percent saturation.

d. Drill and insert thermocouples at 1-in. increments.

e. Place the specimen in a temperature-controlled freezing cabinet (Figure B2), with the bottom of the cabinet open to an ambient temperature of +40° F. (Four specimens can be placed in each cabinet of the type used at CRREL.)

f. Connect the baseplate to a free water supply, with the water surface held level with the upper surface of the specimen.

g. Place surcharge weights equal to the weight of overlying materials in the pavement structure.

h. Pour granulated cork around the specimen or specimens, level with the top, to insure unidirectional downward freezing during the test. As stated previously, cork is preferable to styrofoam beads because of the clinging nature of statically charged foam beads.

i. Freeze the specimen(s) from the top down, advancing the freezing front at a rate of 6 in./day, then thaw the specimen at a rate of 12 in./day.

j. Repeat Step i for a frost penetration rate of 2 in./day, thaw the specimen at the rate of 12 in./day, and repeat again at a frost penetration rate of 0.5 in./day. Do not thaw after the final freezing.

k. Remove the specimen from the freezing cabinet to a cold room at +25° F or lower.

l. Using a carbide-tipped core drill (Figure B4), core four 1.4-in.-diam by 6-in.-long specimens while they are frozen. Chilled air must be used to eject the cuttings. Each 6-in.-long core will provide two triaxial specimens 1.4 by 3 in. in length if the core is not damaged during the coring operation. Care should be taken to obtain at least one triaxial specimen from each core.

m. Cut the specimens to length and machine the ends flat and parallel. At CRREL, a band saw and a lathe are used for these operations. Carbide-tipped tools should be used, and all machining should be conducted in the cold room. Coarse-grained frozen sands are particularly difficult to machine and often require much hand work with rasps and files.

n. Place the machined specimen in a rubber membrane and seal with plastic disks to prevent sublimation during storage. If prolonged periods of storage (2 weeks or longer) are required, the membrane-enveloped specimen should be placed in an airtight container or plastic bag containing snow or ice chips.

SILTY AND CLAYEY GRAVELS

13. The freeze-thaw procedures for cohesive soils, silts, and silty sands should be followed, with the exception that the specimen should be molded and frozen in a right cylinder of the same diameter as that required for the triaxial test (2.5 to 3.0 in.). Thus, no coring operations are required. It should be noted that side restraint during freezing may inhibit heaving, and thus the frost action may be less severe than that experienced in tapered cylinders.

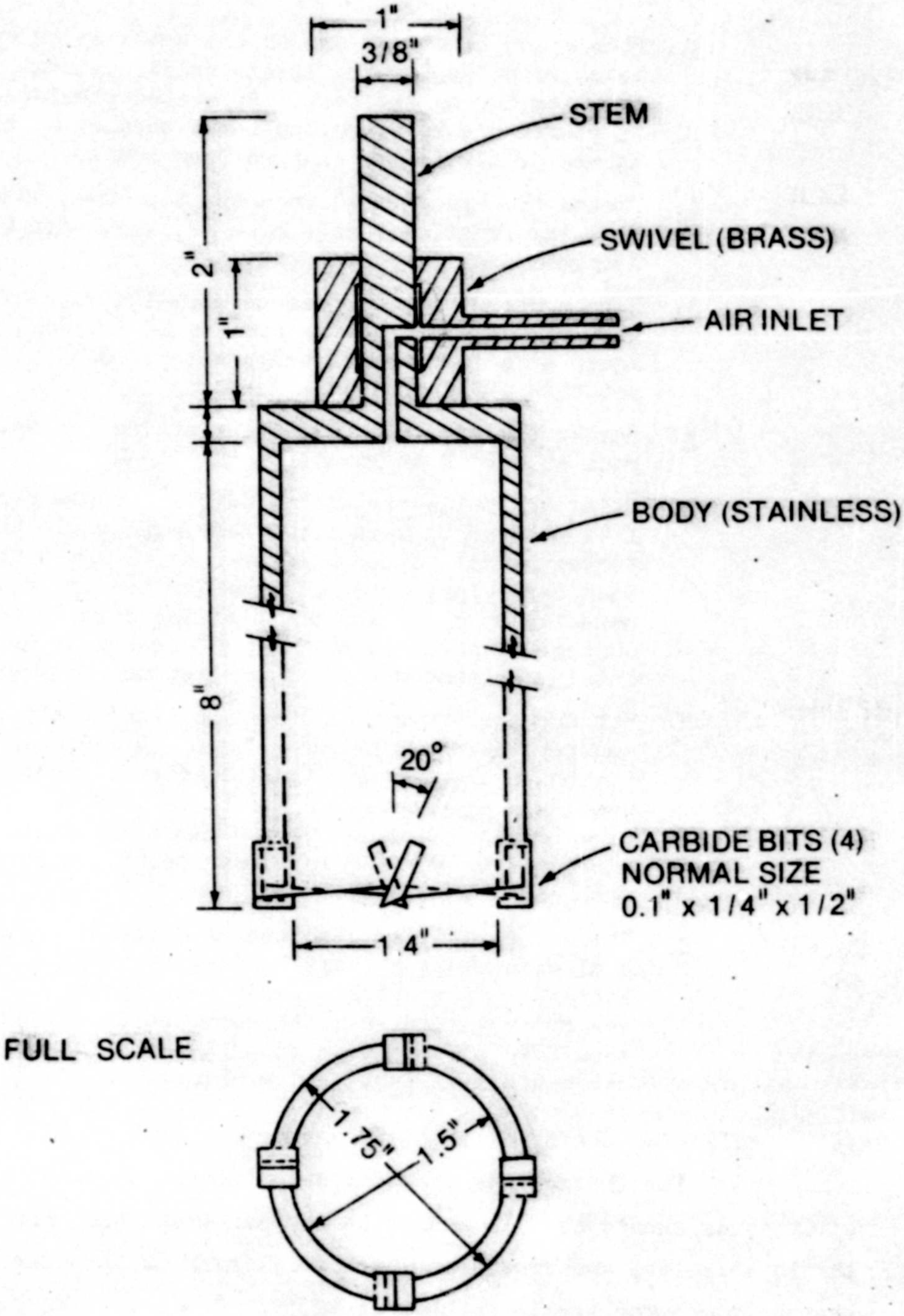


Figure B4. 1.4-in.-ID core drill

TRIAXIAL TEST CELL

14. A triaxial cell suitable for use in resilience testing of soils is shown in Figure B5. It can accommodate either 1.4- or 2.5- to 3.0-in.-diam specimens. This equipment is similar to most standard cells, with the exception of being somewhat larger to facilitate the internally mounted load and deformation measuring equipment and having additional outlets for the electrical leads from the measuring devices. Nitrogen or air is used as the cell fluid.

DEFORMATION MEASUREMENT AND RECORDING

15. The equipment for measurement of axial deformation consists of two linear variable differential transformers (LVDT's) attached to the soil specimen by a pair of clamps like those shown in Figure B6. The clamps are held in tight contact with the specimen by means of the springs shown in the figure. Suitable linear capacity in the LVDT should be provided to allow for highly plastic strains that may occur on thawed soils. The load is measured by a load cell placed on the specimen cap inside the triaxial cell.

16. Use of the type of measuring equipment described above offers several advantages:

<u>a</u>. It is not necessary to reference deformations to the equipment, which deforms during loading.

<u>b</u>. The effect of end-cap restraint on soil response is virtually eliminated.

<u>c</u>. Any effects of piston friction are eliminated by measuring loads inside the triaxial cell.

<u>d</u>. It is not necessary to achieve perfect seating of the end caps by conditioning the specimens by stress repetitions prior to resilience testing.

17. A dual-channel, high-response recorder should be used to record the average of the signals from the two LVDT's and the deviator load.

REPEATED LOADING DEVICE

18. The repeated load source may be any device capable of providing a pulse load of fixed frequency and duration. Simple electromechanical devices, such as an electrical powered cam with static

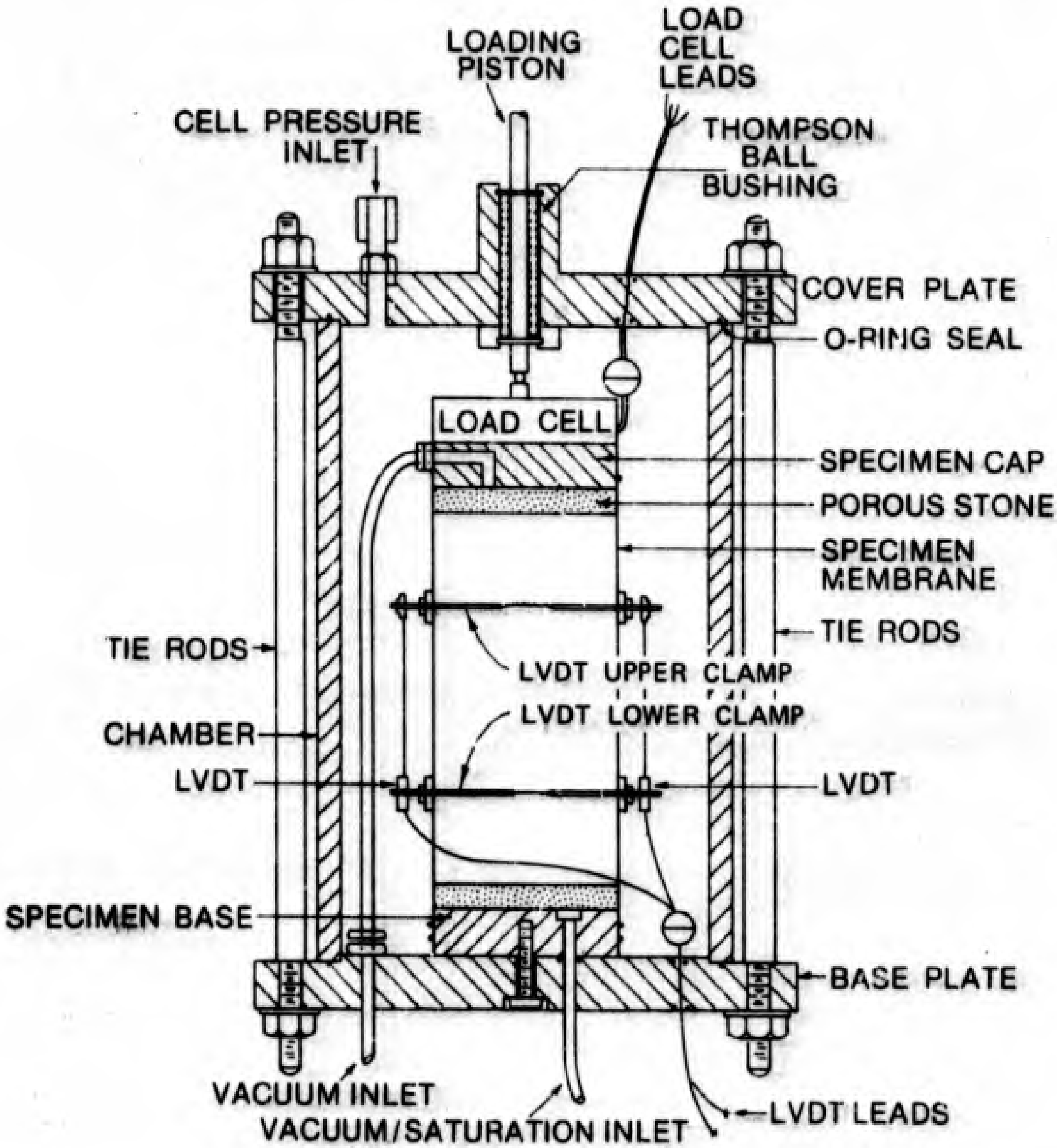


Figure B5. Triaxial cell

weight mechanism, can be used. However, closed-loop electrohydraulic systems are preferable since load duration, frequency, and intensity can be freely selected. A load duration of 0.2 sec and frequency of 20 pulses per minute have been found to be satisfactory for most applications. A square-wave load form is recommended.

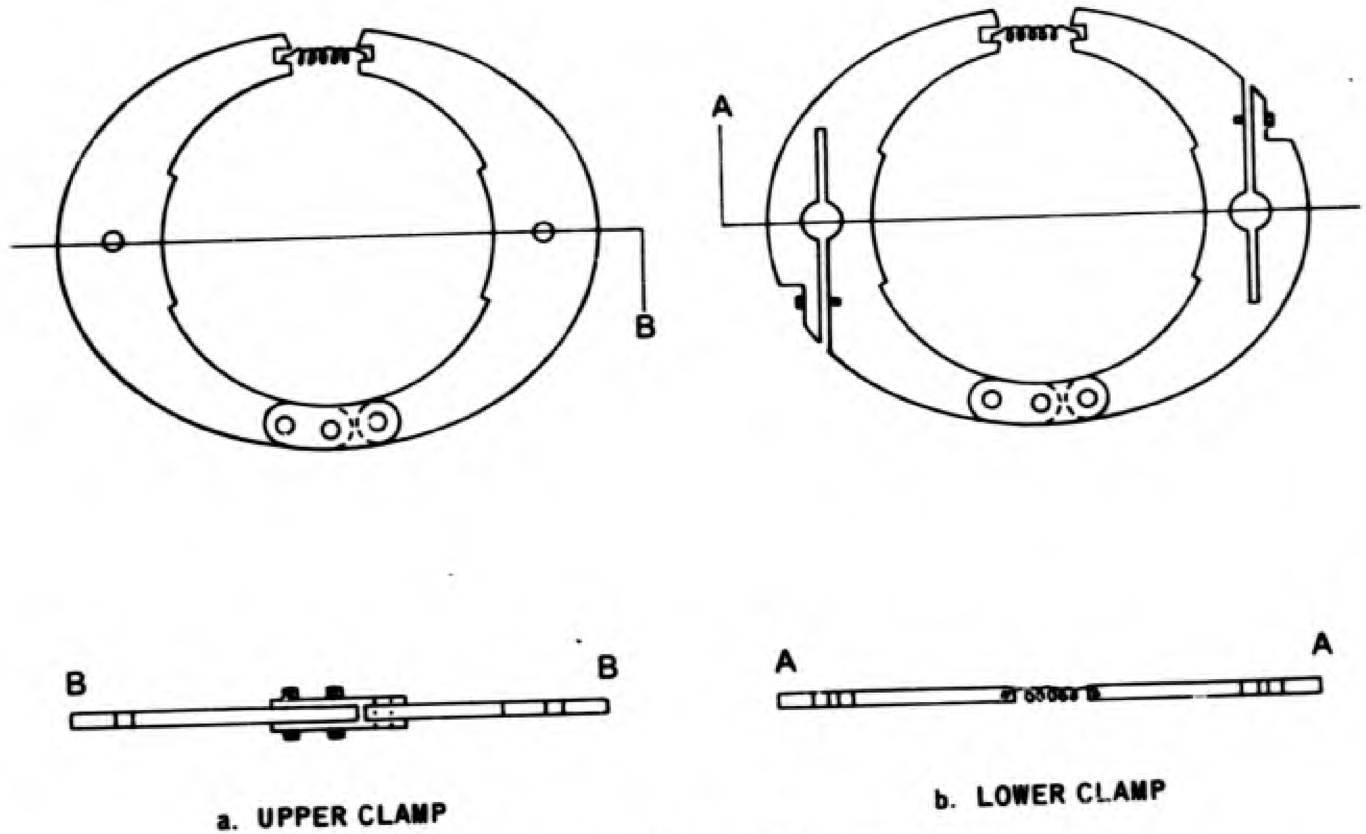


Figure B6. LVDT clamps

19. For testing large-diameter gravel specimens, a 10- to 30-ton-capacity loading machine is required. For testing the small-diameter specimens of sands, silts, and clays, a 1/2-ton-capacity machine is sufficient.

ADDITIONAL EQUIPMENT

20. In addition to the equipment described above, the following items are also used:

- a. Calipers, a micrometer gage, and a steel rule (calibrated to 0.01 in.).
- b. Rubber membranes, 0.01 to 0.25 in. thick.
- c. Rubber O-rings.
- d. A vacuum source with a bubble chamber and regulator.
- e. A back-pressure chamber with pressure transducers.
- f. A membrane stretcher.
- g. Porous stones.

THAWING PROCEDURE

CLEAN SANDS AND GRAVELS

21. The thawing procedure is not applicable, since these materials are to be tested without freezing or thawing.

CLAYS, SILTS, AND CLAYEY OR SILTY SANDS AND GRAVELS

22. The procedure to be used allows no drainage during thawing. The intent of the undrained thawing procedure is to simulate the worst field conditions during thawing when drainage might be restricted by the frozen layer beneath the thawing layers. The procedure is as follows:

- a. Place the frozen specimen on the triaxial baseplate, complete with porous stones, end caps, and rubber membrane (0.005- or 0.01-in. thickness preferred, except for specimens containing gravel, which will require 0.01- to 0.025-in. membranes). The porous stones must be saturated with degassed water and frozen prior to assembly.
- b. Lubricate the outside of the membrane with silicone grease and clamp a rigid, split, confinement jacket around the specimen and end caps, allowing the upper

end cap to be relatively free (a seal must still be maintained) to move under axial load (see Figure B7).

c. Assemble the triaxial cell and move it into place in the testing machine at normal room temperature.

d. Close all drainage lines from the specimen and apply a static axial load equal to the weight of the overlying pavement layers. High loads should be applied in increments.

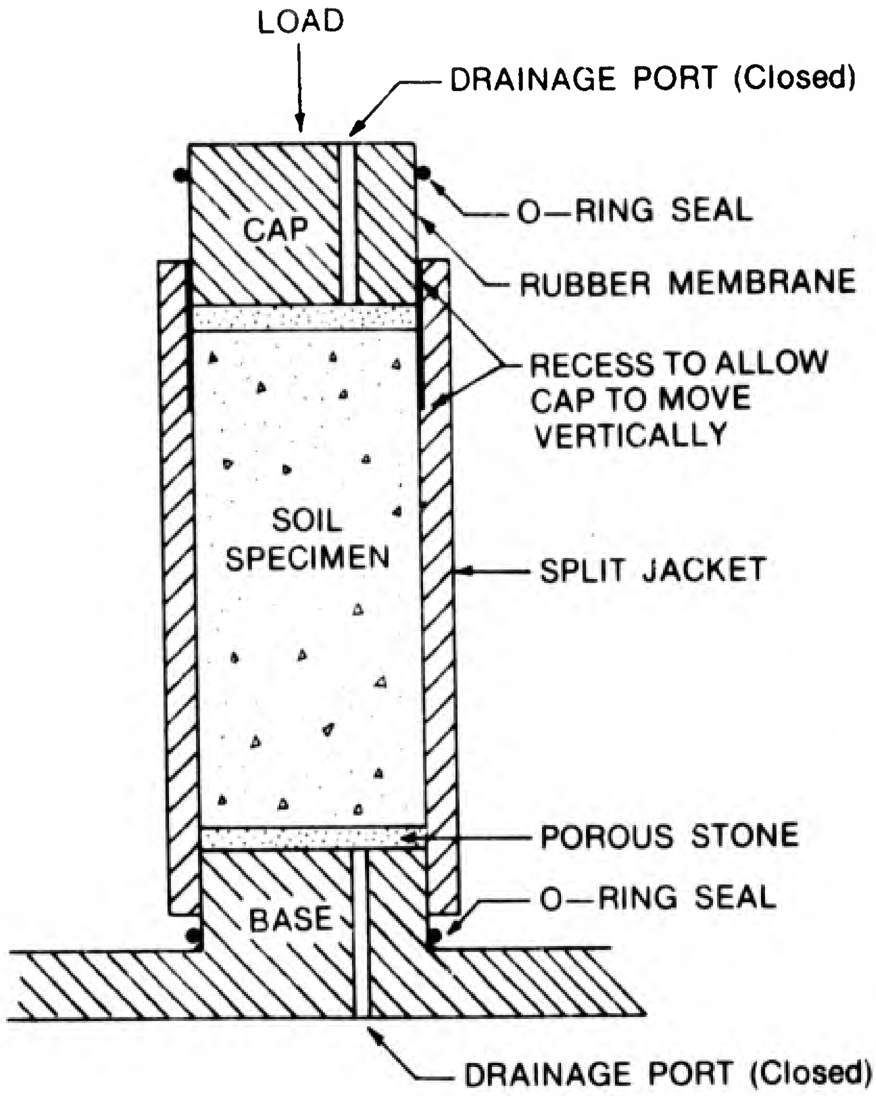


Figure B7. Confinement jacket

e. During thaw, monitor the piston displacement with time using a dial gage or displacement transducer. When deformation of the specimen ceases, the axial load should be removed, the triaxial cell disassembled, and the confinement jacket removed. The drainage valves should be closed throughout this procedure.

f. Assemble and attach the LVDT clamps, connect the LVDT's to the recorder unit, and reassemble the triaxial chamber, as outlined in Paragraph 23 for clean sand and gravel. In no case should specimens that have been frozen and thawed be subjected to the saturation procedures outlined in Paragraph 23, neither by back pressure nor by percolation of de-aired water.

g. At this point, the specimen is ready for Series I resilient testing as described in Paragraphs 24 and 25. In some instances, on removal of the confinement jacket under Step above, the thawed specimen will be found to be too soft to maintain its cylindrical shape. In these cases, Series I tests should not be conducted, and the resilient modulus during thawing without drainage should be assumed to be extremely (and possibly unacceptably) low. The confining jacket should be replaced and an axial load applied equal to the weight of the overlying pavement layers, allowing drainage and maintaining the applied load until full consolidation has occurred. Then the axial load should be reduced to zero, the jacket removed, an all-around confining pressure of 1, 2, or 4 psi applied in accordance with Paragraph 25, full consolidation allowed to occur, and Series II tests begun.

PREPARATION OF SPECIMEN AND PLACEMENT
IN TRIAXIAL CELL (CLEAN SANDS AND GRAVELS)

23. The procedure is as follows:

a. In accordance with procedures specified in EM 1110-2-1906,[45] prepare the specimen and place it on the baseplate, complete with porous stones, cap, and base and equipped with a rubber membrane secured with O-rings. Check for leakage. Maintain the vacuum during placement of the LVDT's. The specimen is now ready to receive the LVDT's.

b. Extend the lower LVDT clamp and slide it carefully down over the specimen to approximately the lower third point of the specimen.

c. Repeat for the upper clamp, placing it at the upper third point. Insure that both clamps lie in horizontal planes.

d. Connect the LVDT's to the recording unit and balance the recording bridges. This will require recorder adjustments and adjustment of the LVDT stems. When a recording bridge balance has been obtained, determine (to the nearest 0.01 in.) the vertical spacing between the LVDT clamps and record this value in Table B1.

e. Place the triaxial chamber in position. Set the load cell in place on the specimen cap.

f. Place the cover plate on the chamber. Insert the loading piston and obtain a firm connection with the load cell.

g. Tighten the tie rods firmly.

h. Slide the assembled apparatus into position under the axial loading device. Bring the loading device to a position where it nearly contacts the loading piston.

i. If the specimen is to be back-pressure saturated, connect the vacuum and saturation units at this time and saturate the specimen following procedures outlined in EM 1110-2-1906.[45] As an alternative, but less effective, procedure, evacuate the specimen under a vacuum of nearly 30 in. of mercury. About 2 hr later, allow de-aired water to percolate slowly upward into the evacuated specimen at a rate of about 1/4 in. per minute. When water flows freely through the top cap, close the water source and leave the specimen in an evacuated state overnight. On the following day, reopen the water source to flush out any remaining bubbles.

j. After saturation has been completed, rebalance the recorder bridge to the load cell and LVDT's.

RESILIENCE TESTING PROCEDURES

24. The resilient modulus of soils not affected by frost action is sometimes determined by applying a series of conditioning stresses to the material before beginning to record deformations, to eliminate initial loading effects. The greatest amount of volume change occurs during the application of the conditioning stresses. No such conditioning stresses should be applied in tests to determine the resilient modulus applicable during the thaw-weakened period, since the reconsolidation that would take place during conditioning would be applicable to a much later phase of recovery of normal summer-fall resilient properties. Therefore, resilient strain should be recorded under the first and succeeding applications of deviator stress. To simulate field

Table B1

Example Data Form for Recording Results of Resilience Tests of Cohesionless Soils

Soil Sample ______________________
____
____

Location ______________________
Sample No. ______________________
Specific Gravity ______________________

Soil Specimen Measurements

Diameter
- Top ______________________
- Middle ______________________
- Bottom ______________________
- Average ______________________

Membrane Thickness ______________________
Net Diameter ______________________
Ht. Specimen + Cap + Base ______________________
Ht. Cap + Base ______________________
Initial Length L_0 ______________________

Soil Specimen Weight

Initial Wt. of Container + Wet Soil - gms ______________________
Final Wt. of Container + Wet Soil - gms ______________________
Wt. Wet Soil Used ______________________

Soil Specimen Volume

Initial Area A_0 in $(inch)^2$ ______________________
Volume $A_0 L_0$ in $(inch)^2$ ______________________
Wet Density pcf ______________________
Water Content - % ______________________
% Saturation ______________________
Dry Density - pcf ______________________
Void Ratio ______________________

Date ______________________

Compaction Method ______________________
____

Vertical Spacing Between LVDT Clamps - inch ______________________

Constants

Vertical LVDT ______________________

Load Cell ______________________

Comments ______________________
____
____

Confining Pressure psi (σ_3)	Load Cell Chart Reading	Deviator Load lbs.	$\sigma_d = \sigma_1 - \sigma_3$ psi	$\frac{\sigma_1}{\sigma_3}$	θ psi	Vertical LVDT Chart Reading	Vertical Deformation inch	ϵ_{R1} in/in	$M_R = \sigma_d/\epsilon_{R1}$ psi

conditions while frost is melting, which may occur more rapidly than reconsolidation, no drainage should be permitted under either confining pressure σ_3 or deviator stress σ_d in Series I tests. To simulate the period of progressive recovery after all frost has melted, Series II tests should be performed allowing full consolidation under σ_3 equal to 1, 2, or 4 psi, whichever most closely approximates the pressure from the overburden only. When the specimen is fully consolidated under that confining stress, additional confining stresses, if any, and repeated deviator stresses should be applied without permitting any drainage.

COHESIVE SOILS

25. The resilient properties of cohesive soils are only slightly affected by the magnitude of the confining pressure σ_3 , and for most applications the effect of different confining pressures can be disregarded. For tests on cohesive soil after freezing and thawing, the confining pressure used should approximate the expected in situ vertical pressure from the overburden only, generally on the order of 1 to 5 psi. A chamber pressure of 1, 2, or 4 psi would be a reasonable value for most testing.

26. Resilient properties of cohesive soil are greatly dependent on the magnitude of the deviator stress σ_d . It is therefore necessary to conduct the test for a range in deviator stress values. Deviator stresses giving principal stress ratios of 2, 4, and 6 are suggested.

- a. After completing the thawing procedure outlined in Paragraph 22, apply the selected confining pressure σ_3 ; in Series I tests, all drainage lines should remain closed.
- b. Set the axial load generator to apply a deviator stress equal to the confining pressure (i.e., a principal stress ratio of 2). Activate the load generator and apply 200 repetitions of this load. Stop the loading.
- c. Still keeping all drainage lines closed, and maintaining the selected confining pressure, apply 200 repetitions of a deviator stress giving a principal stress ratio of 4. Then apply 200 repetitions of a deviator stress giving a principal stress ratio of 6. Stop the loading.
- d. While still maintaining the selected confining pressure, start Series II tests by opening the drainage lines and

letting the specimen consolidate. When consolidation is completed, close the drainage lines and permit no further consolidation under applications of deviator stress.

e. Apply 200 repetitions of a deviator stress giving a principal stress ratio of 2. Stop the loading. Apply 200 repetitions, successively, of deviator stresses giving principal stress ratios of 4 and 6. Stop the loading. Reduce the confining pressure to zero, and dismantle the triaxial cell. Remove the LVDT's and load cell. Use the entire specimen for the purpose of determining the moisture content.

The results of these resilience tests can be presented in the form of a summary table, such as Table B2, and graphically as shown in Figure B8.

COHESIONLESS SOILS AND CLEAN GRANULAR BASE COURSE MATERIALS

27. The resilient modulus M_R of cohesionless soils and granular base materials is dependent upon the magnitude of the confining pressure σ_3 and varies only slightly with changes in the magnitude of the repeated axial stress. Therefore, it is necessary to test cohesionless soils and granular materials over the range of confining stresses expected to exist in the pavement substructure. (The confining pressure is equal to the chamber pressure for dry and wet specimens and is equal to the chamber pressure less the initial back pressure, if any, for saturated specimens.) Accordingly, confining pressures of 1, 2, 4, 10, and 20 psi are suggested. At each confining pressure, tests should be performed at three values of deviator stress corresponding to 1-, 3-, and 5-fold multiples of the confining pressure, giving principal stress ratios of 2, 4, and 6.

28. The procedures for Series I tests are as follows:

a. Close all drainage lines and apply a confining stress σ_3 of 1 psi. Set the axial load generator to apply a deviator stress of 1 psi (i.e., a stress ratio σ_1/σ_3 equal to 2). Activate the load generator and apply 200 repetitions of this load. Stop the loading.

b. Increase σ_3 to 2 psi (no drainage permitted) and set the axial load generator to apply a deviator stress of 2 psi (i.e., maintain a stress ratio of 2). Activate

Table B2

Example Data Form for Recording Results of Resilience Tests of Cohesive Soils

Soil Sample ______________________
____
Location ______________________
Sample No. ______________________
Specific Gravity ______________________

Soil Specimen Measurements

Diameter
- Top ______________________
- Middle ______________________
- Bottom ______________________
- Average ______________________

Membrane Thickness ______________________
Net Diameter ______________________
Ht. Specimen + Cap + Base ______________________
Ht. Cap + Base ______________________
Initial Length L_0 ______________________

Soil Specimen Weight

Initial Wt. of Container + Wet Soil - gms ______________________
Final Wt. of Container + Wet Soil - gms ______________________
Wt. Wet Soil Used ______________________

Soil Specimen Volume

Initial Area A_0 in $(inch)^2$ ______________________
Volume A_0L_0 in $(inch)^3$ ______________________
Wet Density - pcf ______________________
Water Content - % ______________________
% Saturation ______________________
Dry Density - pcf ______________________

Date ______________________

Compaction Method ______________________
____

Vertical Spacing Between LVDT Clamps - inch ______________________
Chamber Pressure - psi ______________________

Constants

Vertical LVDT ______________________

Load Cell ______________________

Comments ______________________
____
____

Load Cell Chart Reading	Deviator Load lbs	σ_d psi	Vertical LVDT Chart Reading	Vertical Defor-mation Inches	ϵ_{R1} in/in	$M_R = \sigma_d/\epsilon_{R1}$ psi

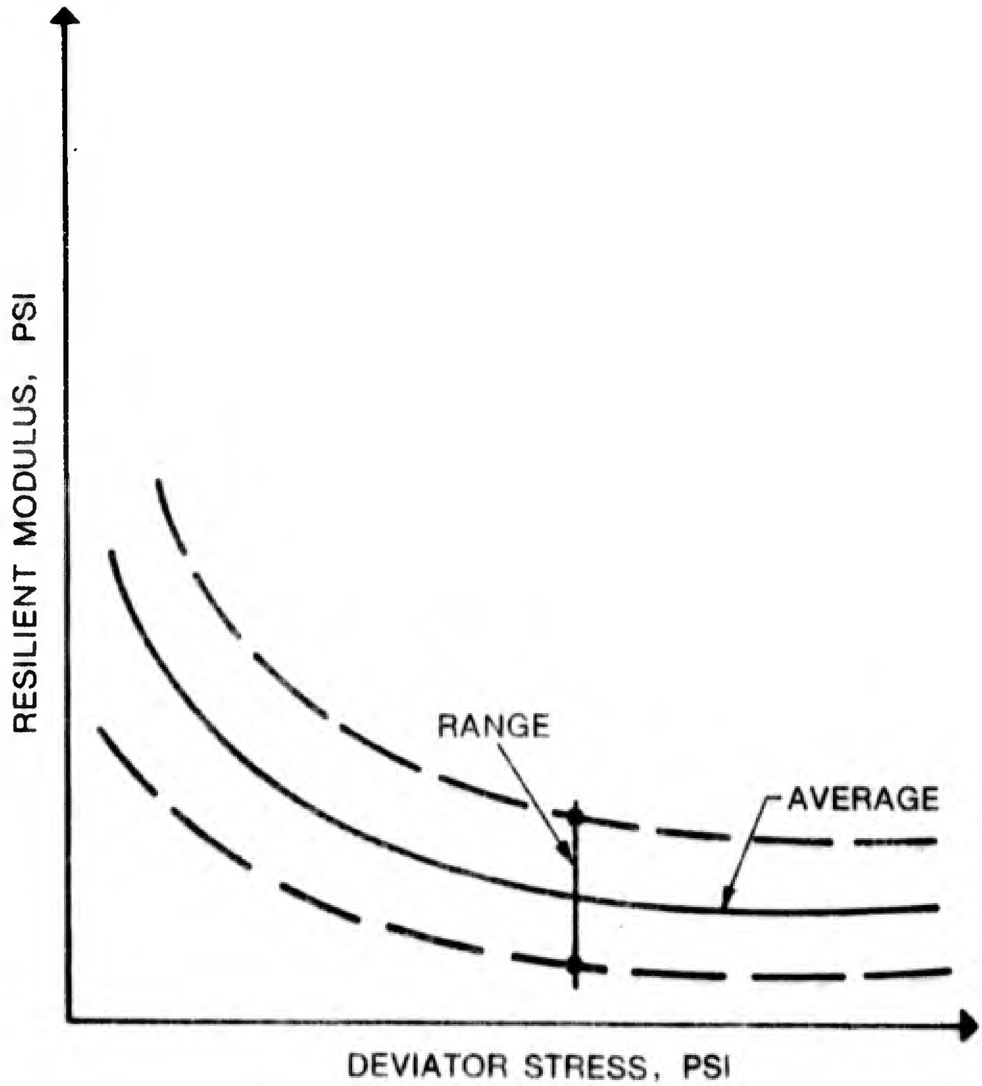


Figure B8. Presentation of results of resilience tests on cohesive soils

the load generator and apply 200 repetitions of this load. Stop the loading. Increase σ_3 successively to 4, 10, and 20 psi (permitting no drainage), applying in each case 200 repetitions of a deviator stress that gives a stress ratio of 2.

c. Repeat Steps a and b, applying a deviator stress giving a stress ratio of 4. Again repeat Steps a and b, applying a deviator stress giving a stress ratio of 6.

29. The procedures for Series II tests are as follows:

a. Decrease σ_3 to 1, 2, or 4 psi, whichever most closely approximates the expected overburden pressure, open drainage lines from the specimens, and permit full consolidation to take place. Then close all drainage lines and apply 200 repetitions of a deviator stress that gives a stress ratio of 2. Then, without permitting further drainage, increase σ_3 successively to levels equal to the remaining higher levels of the series 1, 2, 4, 10, and 20 psi, applying at each value of σ_3 200 repetitions of a deviator stress giving a stress ratio of 2.

b. Decrease σ_3 to the level at which full consolidation was achieved under Step a. Without permitting further drainage, repeat the procedure outlined in Step a, applying deviator stresses in each case giving a stress ratio of 4.

c. Again decrease σ_3 to the level at which full consolidation was achieved under Step a and repeat the procedure in Step a, applying deviator stresses in each case giving a stress ratio of 6.

d. When the test is completed, decrease the back pressure to zero, reduce the chamber pressure to zero, and dismantle the cell. Remove the LVDT clamps, etc. Remove the soil specimen, and use the entire amount of soil to determine the moisture content.

30. Calculations can be performed using the tabular arrangement in Table B1. Individual test results and series results are most readily presented in graphical form, such as shown in Figure B9.

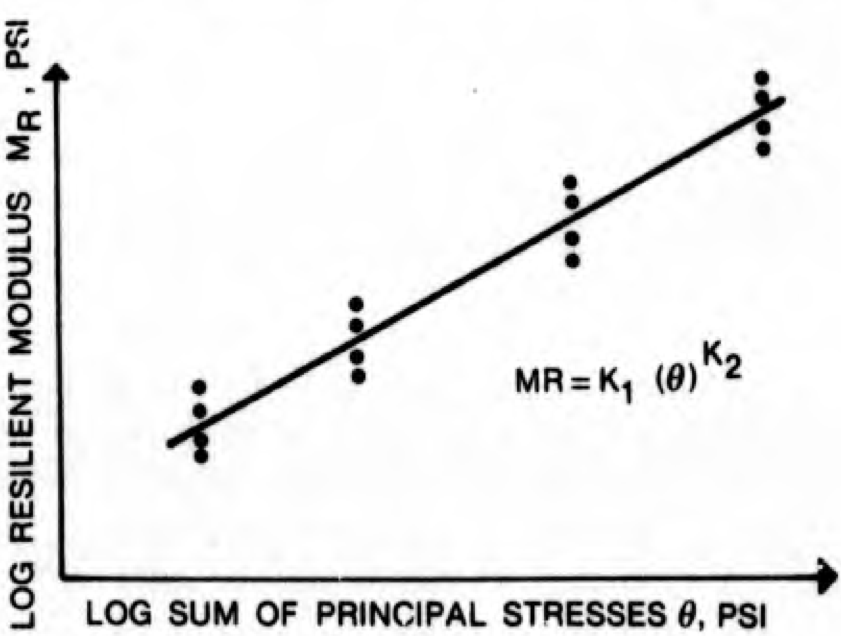


Figure B9. Presentation of results of resilience tests on cohesionless soils

Cohesionless materials such as silts and fine sands and slightly cohesive soils may display the properties illustrated in both Figures B8 and B9. These demonstrate a dependence upon both the cell pressure and the deviator stress. Suitable graphical displays showing such dual dependence would then be more appropriate.

INTERPRETATION OF TEST RESULTS

31. Many of the more critical variables affecting the test results are poorly defined in the current state-of-the-art. These include:

- a. The rate of freezing.
- b. The number of freeze-thaw cycles.
- c. The degree of drainage permitted during final thawing.
- d. The degree of drainage permitted during application of both the all-around confining pressure and the deviator stress.
- e. The extent, if any, to which the specimen should be preconditioned or reconsolidated by deviator stress repetitions prior to recording the particular resilient strain to be used in computing the resilient modulus.

32. The current rudimentary state-of-the-art demands a conservative posture not only in the testing procedures but in the interpretation of results. A problem arises in the selection of a representative modulus to be applied for a period of several weeks or months, and applicable to a layer several inches or feet in thickness, when in reality, the modulus is both time- and space-dependent. During the frost-melting period, the resilient modulus of a soil or an unbound material at a particular level in a pavement substructure is believed to reach a minimum at the time the advancing thaw front reaches that level and immediately thereafter. At this time its condition and behavior are typical of a highly underconsolidated material. The material then begins a gradual process of reconsolidation, which may continue throughout the spring and summer. The rate at which reconsolidation occurs, and its accompanying recovery of normal stress-strain characteristics, depends upon soil properties, proximity and efficacy of drainage layers, and frequency duration and magnitude of wheel loads

applied to the pavement surface. Thus, the resilient modulus at a particular depth varies continuously from the onset of the thaw period. Complicating matters further, the advance of the thaw front to greater depths leaves the material above in various phases of reconsolidation while successive layers of underlying material are reaching their minimum moduli.

33. Two phases of thaw weakening and recovery can be identified. The first begins when the pavement starts to thaw and ends when all frost has left the ground. During this period, the modulus may remain near its minimum level due to impeded downward drainage caused by the still-frozen material below. The results of Series I tests should be used to characterize the soil during this period, calculating the resilient modulus based on the highest resilient strain recorded between the tenth and two hundredth repetitions. Depending upon the soil type, the modulus used for design would be selected at the appropriate level of deviator stress (Figure B8) or the appropriate value of the sum of the principal stresses (Figure B9).

34. The second phase of thaw weakening and recovery begins when all frost leaves the ground and continues until the soils in the pavement substructure have recovered their normal stress-strain properties. In some cohesive soils, the recovery phase probably continues throughout the spring and summer and may even continue until early winter when refreezing begins. In most cases, however, the terminal point of this phase may be generalized as the time at which 80 percent of the normal resilient modulus has been recovered. During the recovery phase, the soil should be characterized by means of Series II tests, calculating the resilient modulus based on the lowest resilient strain recorded between the first and two hundredth repetitions. Depending upon the soil type, the modulus used for design would be selected at the appropriate level of deviator stress (Figure B8) or the appropriate value of the sum of the principal stresses (Figure B9).

APPENDIX C: LABORATORY PROCEDURE FOR DETERMINING THE RESILIENT MODULUS OF SUBGRADE SOILS

1. The objective of this test procedure is to determine a modulus value for subgrade soils by means of resilient triaxial techniques. The test is similar to a standard triaxial compression test, the primary exception being that the deviator stress is applied repetitively and at several stress levels. This procedure allows testing of soil specimens in a repetitive stress state similar to that encountered by a soil in a pavement under a moving wheel load.

DEFINITIONS

2. The following symbols and terms are used in the description of this procedure:

- a. σ_1 = total axial stress.
- b. σ_3 = total radial stress; i.e., confining pressure in the triaxial test chamber.
- c. $\sigma_d = \sigma_1 - \sigma_3$ = deviator stress; i.e., the repeated axial stress in this procedure.
- d. ϵ_1 = total axial strain due to σ_d.
- e. $M_R = \sigma_d/\epsilon_{R_1}$ = resilient modulus.
- f. $\theta = \sigma_1 + 2\sigma_3 = \sigma_d + 3\sigma_3$ = sum of the principal stresses in the triaxial state of stress.
- g. σ_1/σ_3 = principal stress ratio.
- h. Load duration = time interval over which the specimen is subjected to a deviator stress.
- i. Cycle duration = time interval between successive applications of a deviator stress.

SPECIMENS

3. Various diameter soil specimens may be used in this test but the recommended specimen diameter is 2.5 to 3.0 in. or approximately four times maximum aggregate size. The specimen height should be at least twice the diameter. Undisturbed or laboratory molded specimens can be used. Procedures for obtaining undisturbed soil specimens are given in Engineer Manual 1110-2-1907, "Soil Sampling."[48] Methods for

laboratory preparation of molded specimens and for back-pressure saturation of specimens, if required, are presented in EM 1110-2-1906, "Laboratory Soils Testing."[45]

EQUIPMENT

TRIAXIAL TEST CELL

4. A triaxial cell suitable for use in resilience testing of soils is shown in Figure C1. This equipment is similar to most standard cells, with the exceptions of being somewhat larger to facilitate the internally mounted load and deformation measuring equipment and having additional outlets for the electrical leads from the measuring devices. For the type of equipment shown, air or nitrogen is used as the cell fluid.

5. The external loading source may be any device capable of providing a variable load of fixed cycle and load duration, ranging from simple cam-and-switch control of static weights or air pistons to a closed-loop electrohydraulic system. A load duration of 0.2 sec and a cycle duration of 3 sec have been found to be satisfactory for most applications. A square-wave load form is recommended.

DEFORMATION MEASURING EQUIPMENT

6. The deformation measuring equipment consists of linear variable differential transducers (LVDT's) attached to the soil specimen by a pair of clamps. Two LVDT's are used for the measurement of axial deformation. The clamps and LVDT's are shown in position on a soil specimen in Figure C1. Details of the clamps are shown in Figure C2. Load is measured by placing a load cell between the specimen cap and the loading piston as shown in Figure C1.

7. Use of the type of measuring equipment described above offers several advantages:

- a. It is not necessary to reference deformations to the equipment, which deforms during loading.
- b. The effect of end-cap restraint on soil response is virtually eliminated.
- c. Any effects of piston friction are eliminated by measuring loads inside the triaxial cell.

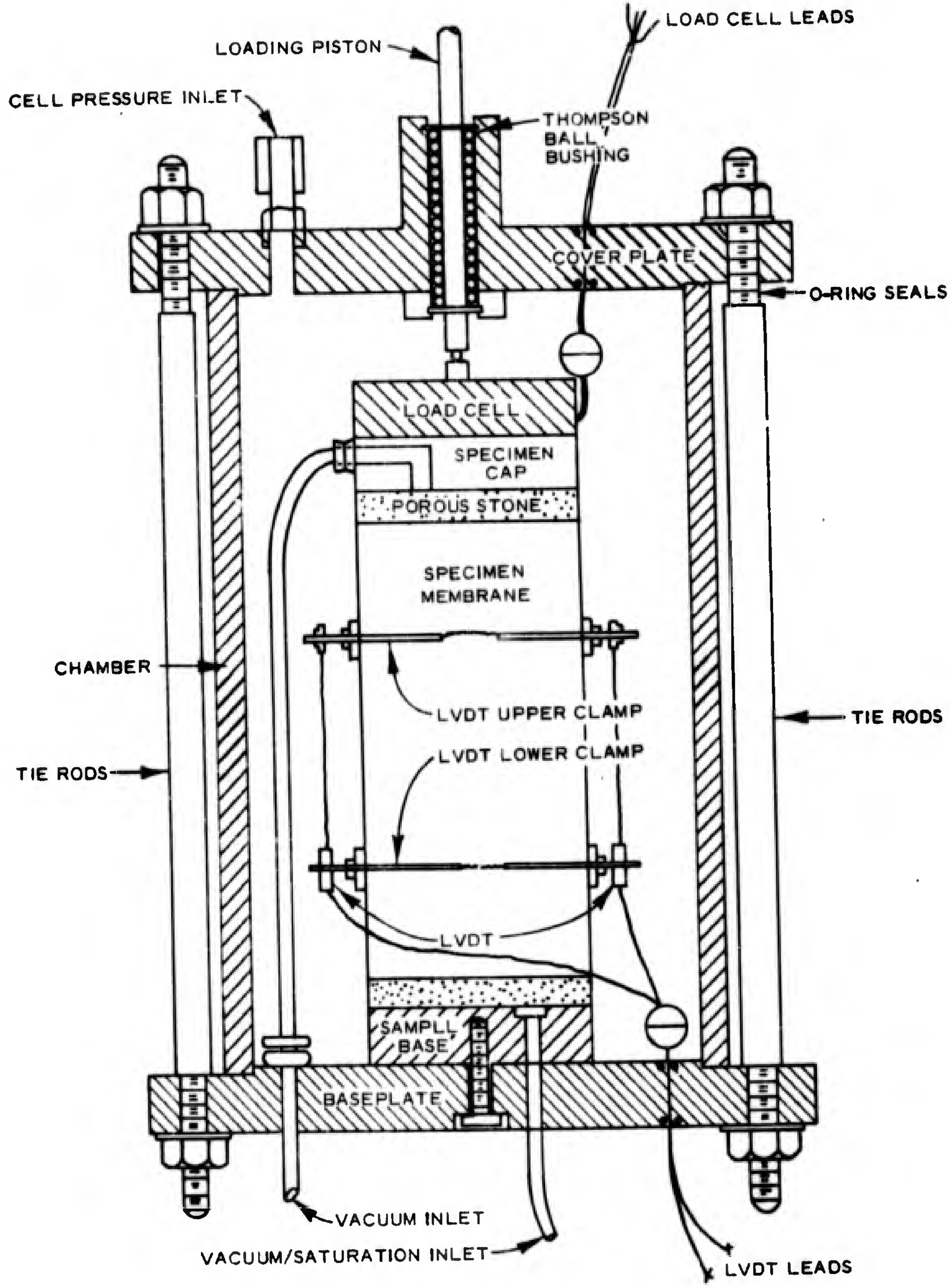


Figure C1. Triaxial cell

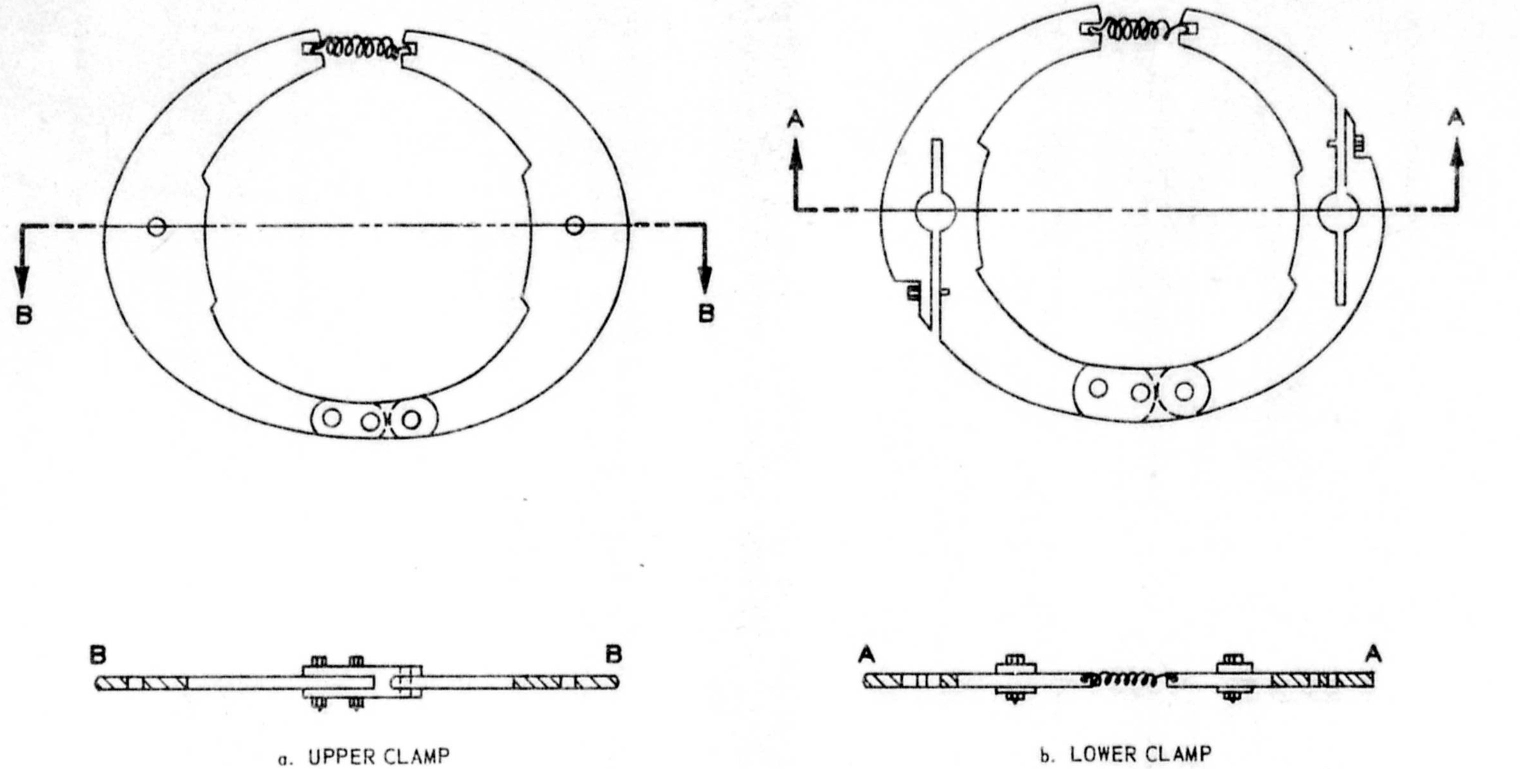


Figure C2. LVDT clamps

8. In addition to the measuring devices, it is also necessary to maintain suitable recording equipment. It is desirable to have simultaneous recording of load and deformation. The number of recording channels can be reduced by wiring the leads from the LVDT's so that only the average signal from each pair is recorded. The introduction of switching and balancing units permits use of a single-chamber recorder. However, this will not permit simultaneous recording.

ADDITIONAL EQUIPMENT

9. In addition to the equipment described above, the following items are also used:

a. A 10- to 30-ton-capacity loading machine.

b. Calipers, a micrometer gage, and a steel rule (calibrated to 0.01 in.).

c. Rubber membranes, 0.01 to 0.025 in. thick.

d. Rubber O-rings.

e. A vacuum source with a bubble chamber and regulator.

f. A back-pressure chamber with pressure transducers.

g. A membrane stretcher.

h. Porous stones.

PREPARATION OF SPECIMENS AND PLACEMENT IN TRIAXIAL CELL

10. The following procedures should be followed in preparing and placing specimens:

a. In accordance with procedures specified in EM 1110-2-1906,[45] prepare the specimen and place it on the baseplate complete with porous stones, cap, and base and equipped with a rubber membrane secured with O-rings. Check for leakage. If back-pressure saturation is anticipated for cohesive soils, procedures indicated in Appendix X to EM 1110-2-1906 for the Q-type triaxial tests should be followed. For purely noncohesive soils, it will be necessary to maintain the vacuum during placement of the LVDT's. The specimen is now ready to receive the LVDT's.

b. Extend the lower LVDT clamp and slide it carefully down over the specimen to approximately the lower third point of the specimen.

c. Repeat this step for the upper clamp, placing it at the upper third point. Insure that both clamps lie in horizontal planes.

<u>d</u>. Connect the LVDT's to the recording unit, and balance the recording bridges. This step will require recorder adjustments and adjustment of the LVDT stems. When a recording bridge balance has been obtained, determine (to the nearest 0.01 in.) the vertical spacing between the LVDT clamps and record this value.

<u>e</u>. Place the triaxial chamber in position. Set the load cell in place on the specimen.

<u>f</u>. Place the cover plate on the chamber. Insert the loading piston, and obtain a firm connection with the load cell.

<u>g</u>. Tighten the tie rods firmly.

<u>h</u>. Slide the assembled apparatus into position under the axial loading device. Bring the loading device to a position in which it nearly contacts the loading piston.

<u>i</u>. If the specimen is to be back-pressure saturated, proceed in accordance with EM 1110-2-1906.

<u>j</u>. After saturation has been completed, rebalance the recorder bridge to the load cell and LVDT's.

RESILIENCE TESTING OF COHESIVE SOILS

11. The resilient properties of cohesive soils are only slightly affected by the magnitude of the confining pressure σ_3 . For most applications, this effect can be disregarded. When back-pressure saturation is not used, the confining pressure used should approximate the expected in situ horizontal stresses. These will generally be on the order of 1 to 5 psi. A chamber pressure of 3 psi is a reasonable value for most testing. If back-pressure saturation is used, the chamber pressure will depend on the required saturation pressure.

12. Resilient properties are highly dependent on the magnitude of the deviator stress σ_d . It is therefore necessary to conduct the tests for a range in deviator stress values. The following procedure should be followed:

<u>a</u>. If back-pressure saturation is not used, connect the chamber pressure supply line and apply the confining pressure (equal to the chamber pressure). If back-pressure saturation is used, the chamber pressure will already have been established.

<u>b</u>. Rebalance the recording bridges for the LVDT's, and

balance the load cell recording bridge.

c. Begin the test by applying 500 to 1000 repetitions of a deviator stress of not more than one-half the unconfined compressive strength.

d. Decrease the deviator load to the lowest value to be used. Apply 200 repetitions of load, recording the recovered vertical deformation at or near the last repetition.

e. Increase the deviator load, recording deformations as in Step d. Repeat over the range of deviator stresses to be used.

f. At the completion of the loading, reduce the chamber pressure to zero. Remove the chamber LVDT's and load cell. Use the entire specimen for the purpose of determining the moisture content.

13. The results of the resilience tests can be presented in the form of a summary table, such as Table C1, and graphically as is shown in Figure C3 for the resilient modulus.

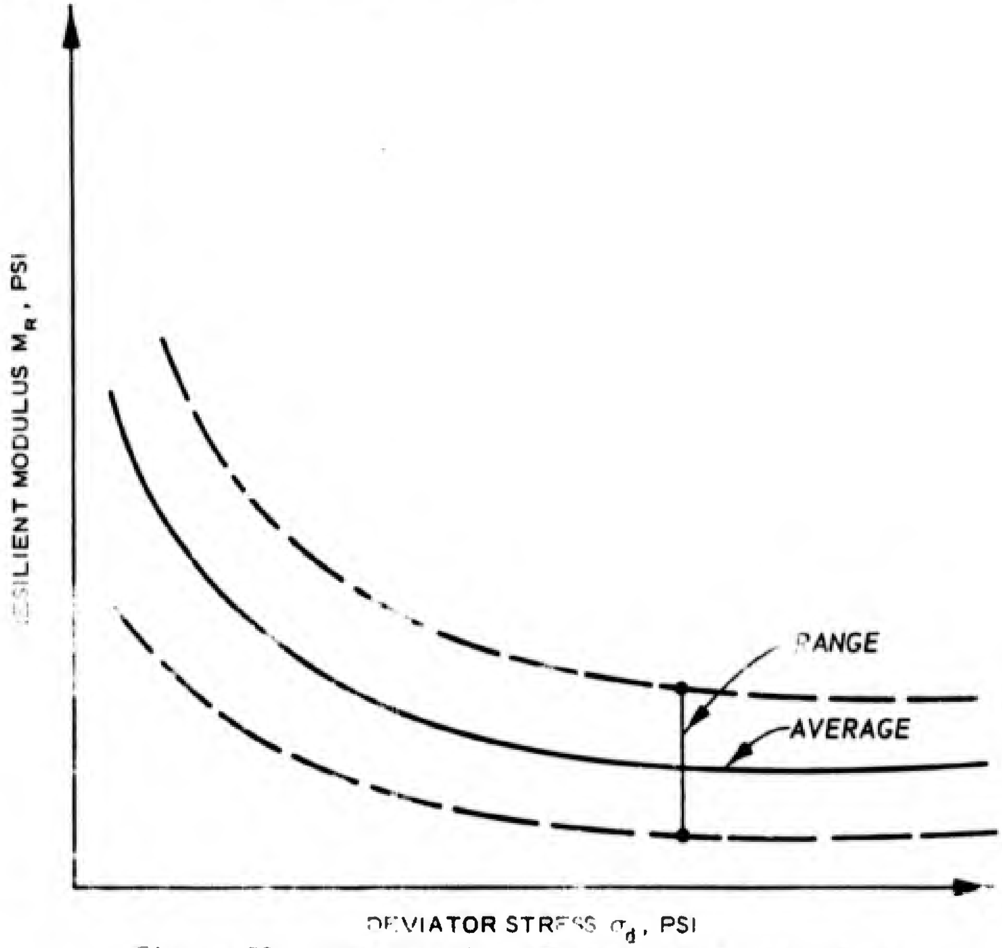


Figure C3. Presentation of results of resilience tests on cohesive soils

Table C1

Example Data Form for Recording Results of Resilience Tests of Cohesive Soils

Soil Sample ____________________

____

Location ____________________
Sample No. ____________________
Specific Gravity ____________________

Soil Specimen Measurements

Diameter Top ____________________
Diameter Middle ____________________
Diameter Bottom ____________________
Average ____________________
Membrane Thickness ____________________
Net Diameter ____________________
Ht. Specimen + Cap + Base ____________________
Ht. Cap + Base ____________________
Initial Length L_0 ____________________

Soil Specimen Weight

Initial Wt. of Container + Wet Soil - gms ____________________
Final Wt. of Container + Wet Soil - gms ____________________
Wt. Wet Soil Used ____________________

Soil Specimen Volume

Initial Area A_0 in $(\text{inch})^2$ ____________________
Volume A_0L_0 in $(\text{inch})^3$ ____________________
Wet Density - pcf ____________________
Water Content - % ____________________
% Saturation ____________________
Dry Density - pcf ____________________

Date ____________________

Compaction Method ____________________

____

Vertical Spacing Between LVDT Clamps - inch ____________________
Chamber Pressure - psi ____________________

Constants

Vertical LVDT ____________________

Load Cell ____________________

Comments ____________________

____

____

Load Cell Chart Reading	Deviator Load lbs	σ_d psi	Vertical LVDT Chart Reading	Vertical Deformation Inches	ϵ_{R1} in/in	$M_R = \sigma_d/\epsilon_{R1}$ psi

RESILIENCE TESTING OF COHESIONLESS SOILS

14. The resilient modulus of cohesionless soils M_R is dependent upon the magnitude of the confining pressure σ_3 and is nearly independent of the magnitude of the repeated axial stress. Therefore, it is necessary to test cohesionless materials over a range of confining and axial stresses. (The confining pressure is equal to the chamber pressure less the back pressure for saturated specimens.) The following procedures should be used for this type of test:

a. Use confining pressures of 5, 10, 15, and 20 psi. At each confining pressure, test at five values of the principal stress difference corresponding to multiples (1, 2, 3, 4) of the cell pressure.

b. Before beginning to record deformations, apply a series of conditioning stresses to the material to eliminate initial loading effects. The greatest amount of volume change occurs during the application of the conditioning stresses. Simulation of field conditions suggests that drainage of saturated specimens should be permitted during the application of these loads but that the test loading (beginning in Step f below) should be conducted in an undrained state.

c. Set the axial load generator to apply a deviator stress of 10 psi (i.e., a stress ratio equal to 3). Activate the load generator and apply 200 repetitions of this load. Stop the loading.

d. Set the axial load generator to apply a deviator stress of 20 psi (i.e., a stress ratio equal to 5). Activate the load generator and apply 200 repetitions of this load. Stop the loading.

e. Repeat as in Step d above maintaining a stress ratio equal to 6 and using the following order and magnitude of confining pressures: 10, 20, 10, 5, 3, and 1 psi.

f. Begin the record test using a confining pressure of 1 psi and an equal value of deviator stress. Record the resilient deformation after 200 repetitions. Increase the deviator stress to twice the confining pressure and record the resilient deformation after 200 repetitions. Repeat until a deviator stress of 4 times the confining pressure is reached (stress ratio of 5).

g. Repeat as in Step f above for each value of confining pressure.

h. When the test is completed, decrease the back pressure to zero, reduce the chamber pressure to zero, and dismantle the cell. Remove the LVDT clamps, etc. Remove the soil specimen, and use the entire amount of soil to determine the moisture content.

15. Calculations can be performed using the tabular arrangement shown in Table C2. Test results should be presented in the form of a plot of log M_R versus log of the sum of the principal stresses as shown in Figure C4.

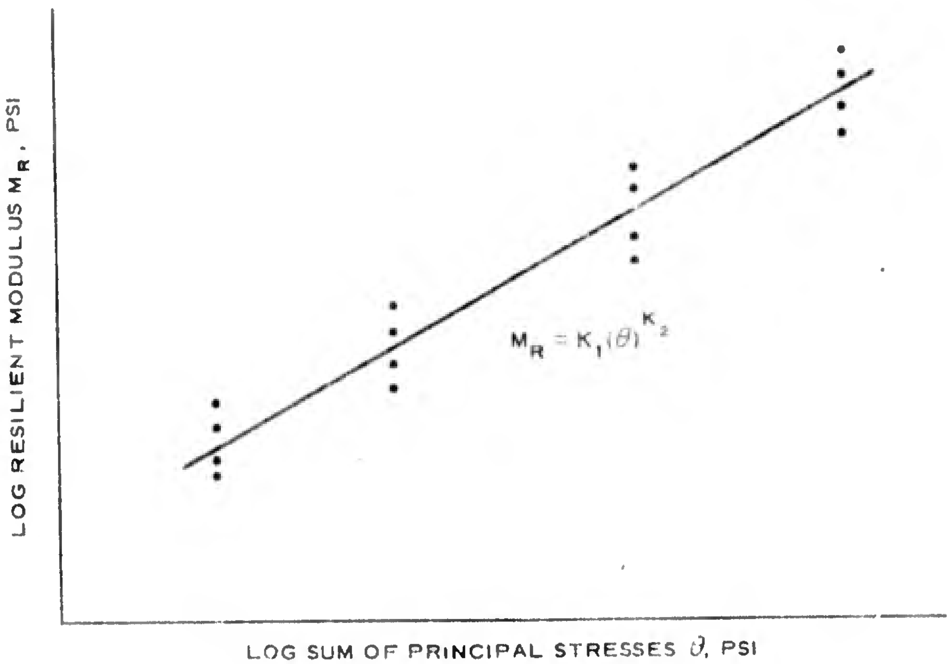


Figure C4. Presentation of results of resilience tests on cohesionless soils

INTERPRETATION OF TEST RESULTS

16. As previously indicated, test results for cohesive soils are presented in the form of a plot of resilient modulus M_R versus deviator stress σ_d. Normally for cohesive soils, the test results will indicate that the resilient modulus decreases rapidly with increases in deviator stress. Thus, selection of a resilient modulus from the laboratory test results requires an estimate of the deviator stress at the top of the subgrade with respect to the design aircraft. For a properly designed pavement, the deviator stress at the top of the

Table C2

Example Data Form for Recording Results of Resilience Tests of Cohesive Soils

Soil Sample ______________________

____

Location ______________________

Sample No. ______________________

Specific Gravity ______________________

Soil Specimen Measurements

Diameter
- Top ______________________
- Middle ______________________
- Bottom ______________________
- Average ______________________

Membrane Thickness ______________________

Net Diameter ______________________

Ht. Specimen + Cap + Base ______________________

Ht. Cap + Base ______________________

Initial Length L_0 ______________________

Soil Specimen Weight

Initial Wt. of Container + Wet Soil - gms ______________________

Final Wt. of Container + Wet Soil - gms ______________________

Wt. Wet Soil Used ______________________

Soil Specimen Volume

Initial Area A_0 in $(inch)^2$ ______________________

Volume A_0L_0 in $(inch)^2$ ______________________

Wet Density pcf ______________________

Water Content - % ______________________

% Saturation ______________________

Dry Density - pcf ______________________

Void Ratio ______________________

Date ______________________

Compaction Method ______________________

____

Vertical Spacing Between LVDT Clamps - inch ______________________

Constants

Vertical LVDT ______________________

Load Cell ______________________

Comments ______________________

____

____

Confining Pressure psi (σ_3)	Load Cell Chart Reading	Deviator Load lbs.	$\sigma_d = \sigma_1 - \sigma_3$ psi	$\frac{\sigma_1}{\sigma_3}$	θ psi	Vertical LVDT Chart Reading	Vertical Deformation inch	ϵ_{R1} in/in	$M_R = \sigma_d/\epsilon_{R1}$ psi

subgrade will primarily be a function of the subgrade modulus and the design traffic level. Shown in Figure C5 are relationships between deviator stress at the top of the subgrade and applicable subgrade modulus values determined from an analysis of the pavement sections described in the main text of this report. The relationships shown in Figure C5 were determined using a layered elastic pavement model with the modulus values as input parameters and the deviator stress values as computed responses. Thus these relationships are essentially limiting criteria. Relationships are shown for 1,200, 6,000, and 25,000 annual departures. To determine the appropriate modulus value to use in the performance model, the test results from the resilient modulus tests on the laboratory specimens are superimposed on the appropriate relationship from Figure C5, and the design modulus value is taken from the intersection of the plotted functions.

17. For example, assume a design problem involving an airport at which the predominant operational aircraft has a dual-tandem main gear assembly and for which the design life is 6000 annual departures. In Figure C6 is shown a plot of the relationship taken from Figure C5 superimposed on test results from a laboratory resilient modulus test. For this particular design, it can be seen that a subgrade modulus value of 9000 psi would be used.

18. For cohesionless soils, laboratory test results are presented in the form of a plot of resilient modulus versus the first stress invariant, i.e., sum of the principal stress θ . For cohesionless soils, this relationship is generally linear in form on a log-log plot, with the resilient modulus being directly proportional to the sum of the principal stresses. Selection of a specific resilient modulus value for use in the design model requires an estimate of the sum of the principal stresses at the top of the subgrade. Since a cohesionless material is involved, the influence of both applied stresses and estimated overburden stresses from the pavement structure must be considered. In Figure C7, a relationship is shown between the pavement thickness and the sum of the principal stresses at the top of the subgrade due to overburden. In Figure C8, relationships are shown between the subgrade

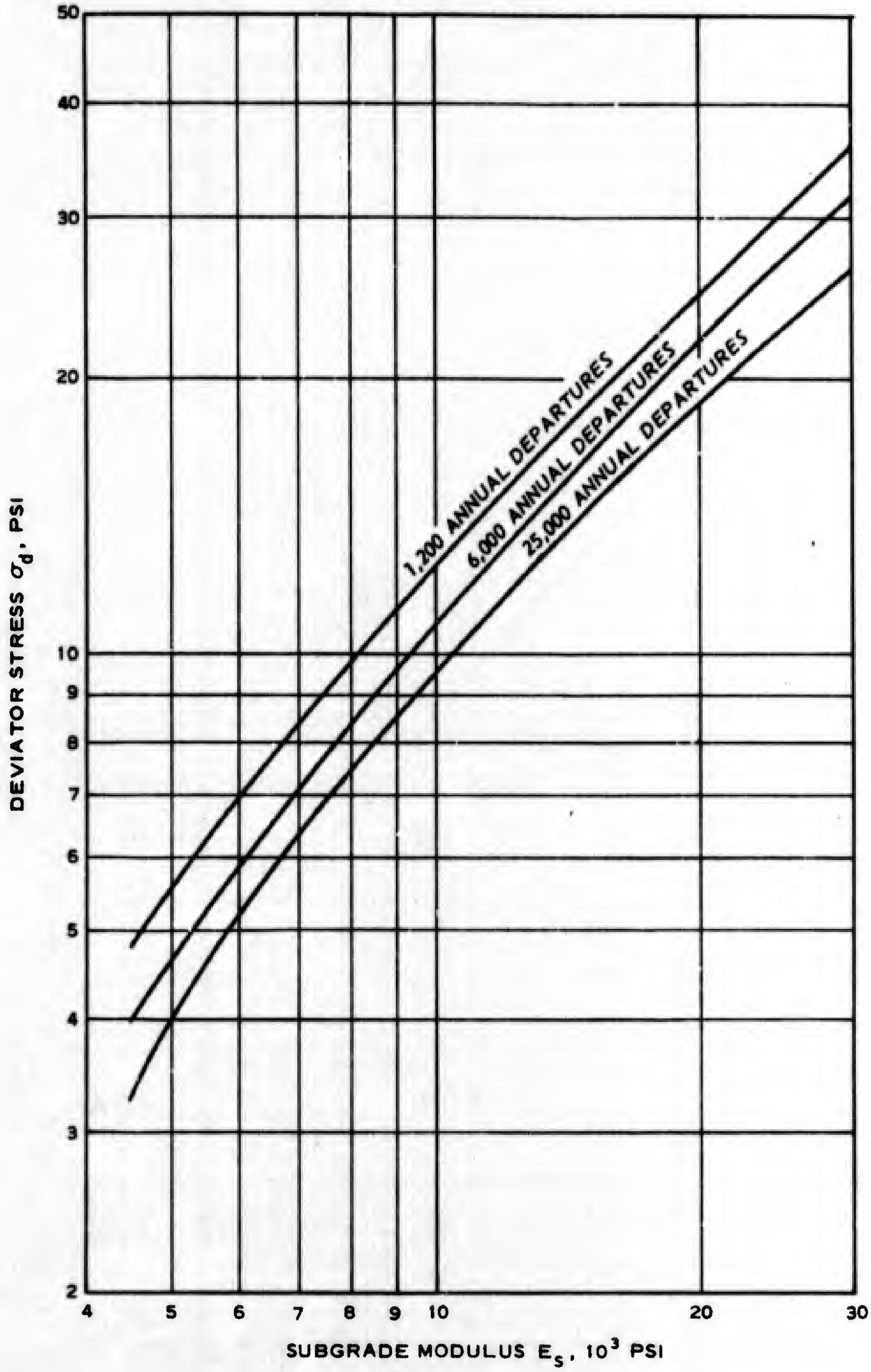


Figure C5. Estimated deviator stress at top of subgrade

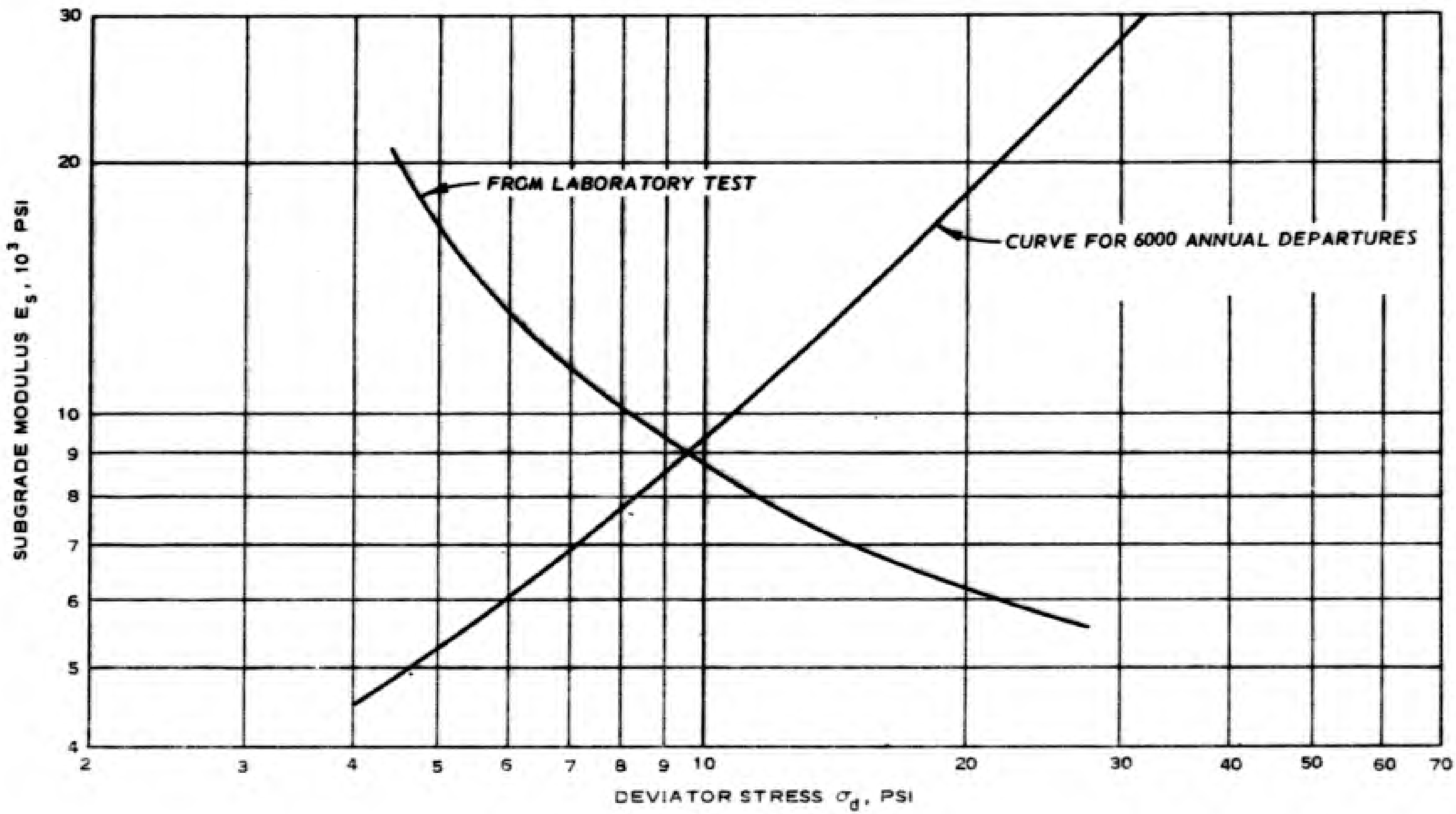


Figure C6. Determination of subgrade modulus for cohesive soils

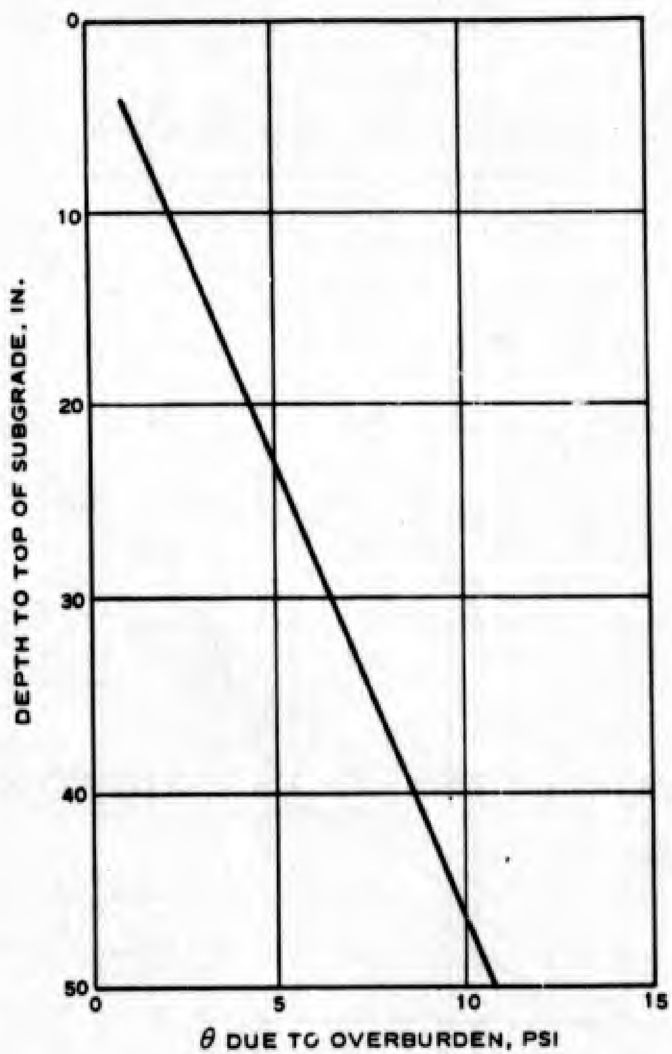


Figure C7. Relationship for estimating θ due to overburden

modulus and limiting values of the sum of the principal stresses due to applied force. For each figure, relationships are shown for 1,200, 6,000, and 25,000 annual departures. Using the value of the estimated pavement thickness, that part of the total sum of the principal stresses due to overburden can be obtained from Figure C7. The applicable relationship from Figure C8 is then selected and adjusted to include the influence of overburden by increasing all values of the principal stress sum by the value obtained from Figure C7. Thus, a new limiting

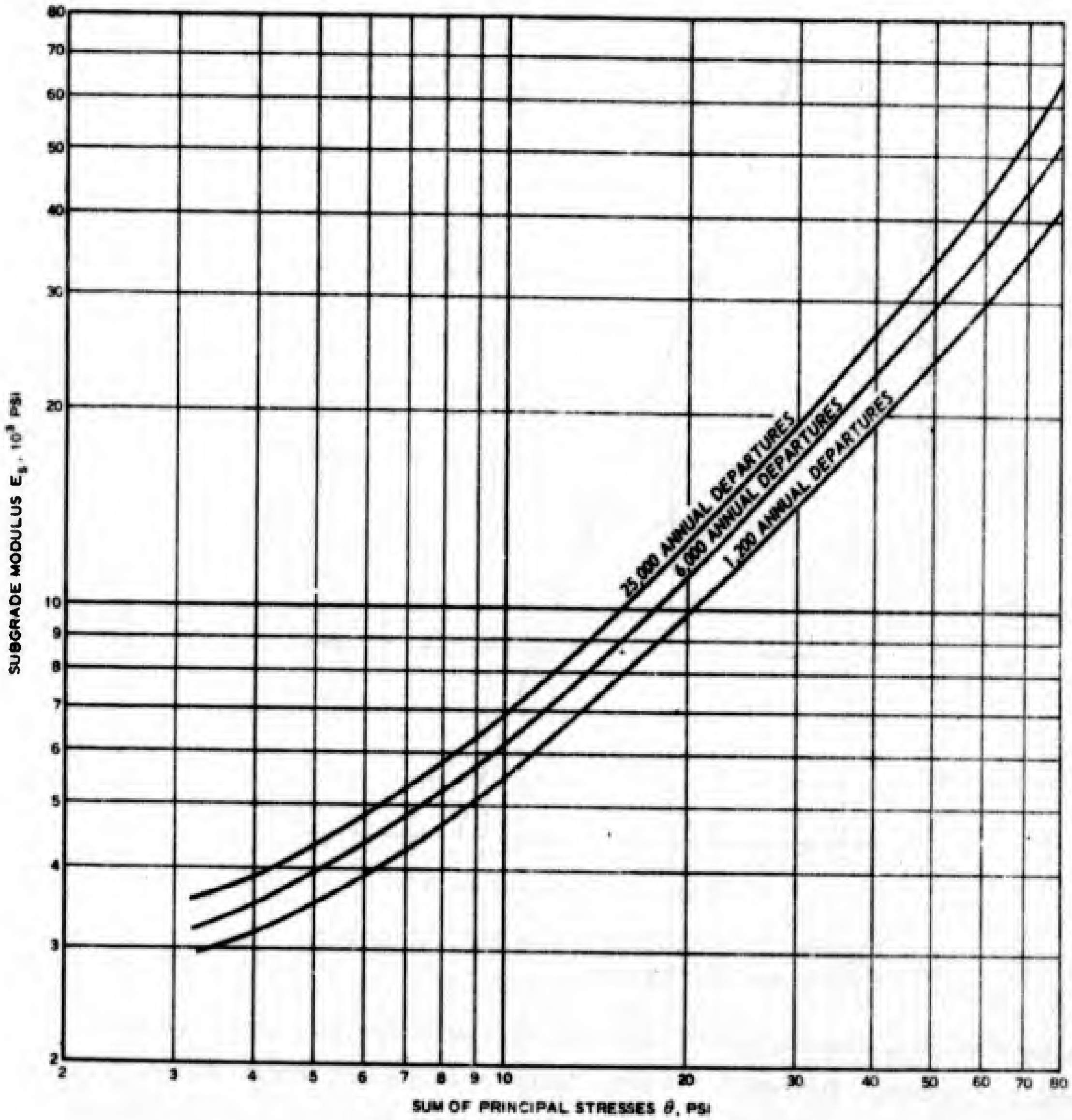


Figure C8. Estimated θ at top of subgrade relationship is obtained and replotted. The results of the laboratory modulus test are superimposed on the plot, and the design subgrade modulus values are taken at the intersection of these relationships.

19. As an example, assume a design problem involving a pavement having an estimated initial thickness of 30 in. The design aircraft has a dual-wheel main gear assembly, and the design life is for 6000 annual departures. From Figure C7, the value of the sum of the principal stresses due to overburden is 6.5 psi. Using the 6000 annual departures curve from Figure C8, the value obtained from Figure C7 is added to all values of the sum of the principal stresses indicated in the relationship and the adjusted curve is replotted (Figure C9). The result of adjusting the original relationship is to shift it to the right of its original position. In Figure C9, the results of laboratory resilient modulus tests on specimens of the subgrade soil are also shown. From the intersection of these two relationships, a design modulus M_R of 15,000 psi is determined.

20. In some situations, the laboratory curve may not converge with the limiting stress-modulus relationship within the range of values indicated. Obviously, two possibilities are involved in this situation: the laboratory relationships could plot above or below the limiting criteria curve. In the former case, since all values of the sum of the principal stresses indicated by the laboratory curve would exceed the stress criteria within the region under consideration, the value of 30,000 psi should be used for the subgrade modulus. In the latter case, the initial design thickness value should be increased and the limiting criteria curve readjusted until convergence with the laboratory relationship is obtained.

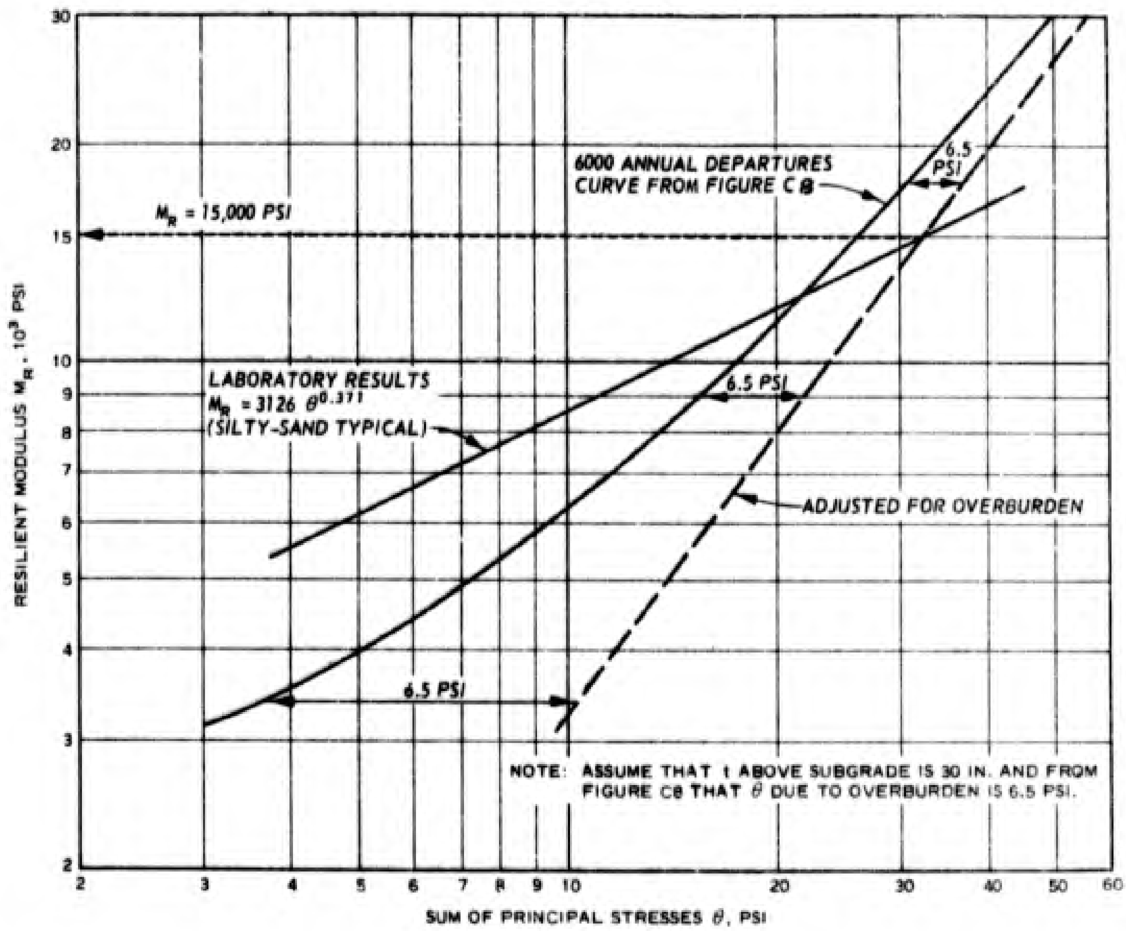


Figure C9. Selection of M_R for silty-sand subgrade with estimated thickness of 30 in. for 6000 annual departures of dual-tandem gear

APPENDIX D: PROCEDURE FOR PREPARATION OF BITUMINOUS CYLINDRICAL SPECIMENS

SCOPE

1. This procedure describes the preparation of cylindrical specimens of bituminous paving mixture suitable for dynamic modulus testing. The procedure is intended for dense-graded bituminous concrete mixtures containing up to 1-in. maximum-size aggregate.

APPLICABLE STANDARDS

2. The following American Society for Testing and Materials (ASTM) standards are applicable to this procedure:

   a. ASTM Designation: D 1559-71, "Standard Method of Test for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus."[26]

   b. ASTM Designation: D 1560-65, "Standard Method of Test for Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus."[26]

   c. ASTM Designation: D 1561-65, "Preparation of Test Specimens of Bituminous Mixtures by Means of California Kneading Compactor."[26]

SPECIMENS

3. Approximately 4000 g of bituminous mixture should be prepared as specified by Method D 1560-65. Cylindrical specimens should be 4 in. in diameter by 8 in. in height.

APPARATUS

4. The apparatus used in preparing the specimens should be as specified by Method D 1561-65, except that steel molding cylinders with 1/4-in. wall thickness having an inside diameter of 4 in. and height of 10 in. should be used.

PROCEDURE

5. The compaction temperature for the bituminous mixture should be as specified by Method D 1561-65. As the first step in molding specimens, heat the compaction mold to the same temperature as the mix. Next, place the compaction mold in position in the mold holder and insert a paper disk 4 in. in diameter to cover the baseplate of the mold

holder. Weigh out one-half of the required amount of bituminous mixture for one specimen at the specified temperature and place uniformly in the insulated feeder trough which has been preheated to the compaction temperature for the mixture. By means of the variable transformer controlling the heater, maintain the compactor foot sufficiently hot to prevent the mixture from adhering to it. By means of a paddle of suitable dimensions to fit the cross section of the trough, push 30 approximately equal portions of the mixture continuously and uniformly into the mold while 30 tamping blows at a pressure of 250 psi are applied. Immediately place the remaining one-half of the mixture uniformly in the feeder trough. Push 30 approximately equal portions of the mixture continuously and uniformly into the mold while 30 tamping blows at a pressure of 250 psi are applied.*

6. Immediately after compaction with the California kneading compactor, apply a static load to the specimen using a compression testing machine. Apply the load by the double-plunger method in which metal followers are employed as free-fitting plungers on the top and bottom of the specimen. Apply the load on the specimen at a rate of 0.5 in. per minute until an applied pressure of 1000 psi is reached. Release the load immediately. After the compacted specimen has cooled sufficiently so that it will not deform on handling, remove it from the mold. Place the specimen on a smooth flat surface and allow to cool to room temperature.**

---

* If sandy or unstable material is involved and there is undue movement of the mixture under the compactor foot, reduce the compaction temperature and compactor foot pressure until kneading compaction can be accomplished.

** Cylindrical specimens will have approximately the same bulk specific gravity as specimens prepared as specified by Method D 1559-71 and by Method D 1561-65.

APPENDIX E: LABORATORY PROCEDURE FOR DETERMINING THE DYNAMIC MODULUS OF BITUMINOUS CONCRETE MIXTURES

1. The purpose of this procedure is to determine dynamic modulus values of bituminous concrete mixtures. The procedure described covers a range of both temperature and loading frequency. The minimum recommended test series consists of testing at 40°, 70°, and 100° F at loading frequencies of 2 and 10 Hz for each temperature. The method is applicable to bituminous paving mixtures similar to Mixes 3A, 4A, 5A, 6A, and 7A as defined by American Society for Testing and Materials (ASTM) Specification D 1663-67.[26]

APPLICABLE STANDARDS

2. The following ASTM standards are applicable to this procedure:

   a. ASTM Designation: C 617-71, "Capping Cylindrical Concrete Specimens."[49]

   b. ASTM Designation: D 1559-71, "Standard Method of Test for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus."[26]

   c. ASTM Designation: D 1561-65, "Preparation of Test Specimens of Bituminous Mixtures by Means of California Kneading Compactor."[26]

   d. ASTM Designation: D 1663-67, "Standard Specification for Hot-Mixed, Hot-Laid Asphalt Paving Mixtures."[26]

SUMMARY OF PROCEDURE

3. The dynamic modulus test is run by applying a sinusoidal (haversine) axial compressive stress to a specimen of bituminous concrete at a given temperature and loading frequency. The resulting recoverable axial strain response of the specimen is measured and used to calculate the dynamic modulus.

DEFINITIONS

4. The following terms are used in this procedure:

   a. Dynamic modulus. The absolute value of the complex modulus which defines the elastic properties of a linear viscoelastic material subjected to a sinusoidal loading.

   b. Complex modulus. A complex number which defines the relationship between stress and strain for a linear viscoelastic material.

c. Linear material. A material whose stress-to-strain ratio is independent of the loading stress applied.

APPARATUS

5. An electrohydraulic testing machine with a frequency generator capable of producing a haversine wave form has proven to be most suitable for use in dynamic modulus testing. The testing machine should have the capability of applying loads over a range of frequencies from 0.1 to 20 Hz and stress levels up to 100 psi.

6. The temperature control system should be capable of a temperature range of 32° to 120° F ± 1° F. The temperature chamber should be large enough to hold six specimens.

7. The measurement system should consist of a two-channel recorder, stress and strain measuring devices, and suitable signal amplification and excitation equipment. The measurement system should have the capability for determining loading up to 3000 lb from a recording with a minimum sensitivity of 2 percent of the test load per millimetre of chart paper. This system should also be capable of use in determining strains over a range of full-scale recorder outputs from 300 to 5000 microunits of strain. At the highest sensitivity setting, the system should be able to display 4 microunits of strain or less per millimetre on the recorder chart.

8. The recorder amplitude should be independent of frequency for tests conducted up to 20 Hz.

9. The values of axial strain should be measured by bonding two wire strain gages* at midheight opposite each other on the specimens. The gages are wired in a Wheatstone bridge circuit with two active gages on the test specimen and two temperature-compensating gages on an unstressed specimen exposed to the same environment as the test specimen. The temperature-compensating gages should be at the same position on the specimen as the active gages. The sensitivity and type of measurement device should be selected to provide the strain readout required in Paragraph 7.

---

* The Baldwin Lima Hamilton SR-4 Type A-1S 13 strain gage has been found satisfactory for this purpose.

10. Loads should be measured with an electronic load cell meeting requirements for load and stress measurements in Paragraph 7.

11. A hardened steel disk with a diameter equal to that of the test specimen should be used to transfer the load from the testing machine to the specimen.

SPECIMENS

12. The laboratory molded specimens should be prepared according to **Appendix D**. A minimum of three specimens is required for testing. The molding procedure is as follows: Cap all specimens with a sulfur mortar meeting Method C 617-71 requirements prior to testing. Bond the strain gages with epoxy cement* to the sides of the specimen near midheight in position to measure axial strains. Wire the strain gages as required in Paragraph 9, and attach suitable lead wires and connectors.

PROCEDURE

13. Place test specimens in a controlled temperature cabinet, and bring them to the specified test temperature.**

14. Place a specimen in the loading apparatus, and connect the strain gage wires to the measurement system. Put the hardened steel disk on top of the specimen and center both under the loading apparatus. Adjust and balance the electronic measuring system as necessary.

15. Apply the haversine loading to the specimen without impact and with loads varying between 0 and 35 psi for each load application for a minimum of 30 sec and not exceeding 45 sec at temperatures of 40°, 70°, and 100° F and at loading frequencies of 2 Hz for taxiway design and 10 Hz for runway design.†

---

* Baldwin Lima Hamilton EPY 150 Epoxy Cement has been found satisfactory for this purpose. On specimens with large-size aggregate, care must be taken so that the gages are attached over areas between the aggregate faces.

** A dummy specimen with a thermocouple in the center can be used to determine when the desired test temperature is reached.

† If excessive deformation (greater than 2500 microunits of strain) occurs, reduce the maximum loading stress level to 17.5 psi.

16. Test three specimens at each temperature and frequency condition twice. Start at the lowest temperature and repeat the test at the next highest temperature. Bring the specimens to the specified test temperature before each test is commenced.

17. Monitor both the loading stress and the axial strain during the test. Increase the recorder chart speed so that one cycle covers 1 to 2 cm of chart paper for five to ten repetitions before the end of the test.

18. Complete the loading for each test within 2 min from the time specimens are removed from the temperature control cabinet.*

CALCULATIONS

19. Measure the average amplitude of the load and the strain over the last three loading cycles to the nearest 1/2 mm. Calculate the loading stress σ_o using the equation

$$\sigma_o = \frac{H_1 L}{H_2 A} \qquad \text{(E1)}$$

where

H_1 = measured height of load

H_2 = measured chart height

L = full-scale load amplitude determined by settings on the recording equipment

A = cross-sectional area of the test specimen

Calculate the recoverable axial strain ε_o using the equation

$$\varepsilon_o = \frac{H_3 S}{H_4} \qquad \text{(E2)}$$

where

H_3 = measured height of recoverable strain

H_4 = measured chart height

S = full-scale strain amplitude determined by settings on the recording equipment

Calculate the dynamic modulus using the equation

* The 2-min testing time limit is waived if loading is conducted within a temperature control cabinet meeting requirements in Paragraph 6.

$$|E^*| = \frac{\sigma_o}{\varepsilon_o} \qquad \text{(E3)}$$

where

σ_o = axial loading stress, psi

ε_o = recoverable axial strain, in./in.

20. Report the average dynamic modulus at temperatures of 40°, 70°, and 100° F for each loading frequency at each temperature.

APPENDIX F: PROCEDURE FOR ESTIMATING THE MODULUS OF ELASTICITY OF BITUMINOUS CONCRETE

1. The procedure for estimating the modulus of elasticity of bituminous concrete presented here is based on relationships developed by Shell. Parameters needed for input into this method are:

   a. Ring-and-ball softening point, in degrees Celcius, of the bituminous material used in the mix in accordance with American Society for Testing and Materials (ASTM) Designation: D 36-70.[26]

   b. Penetration of the bituminous material, in 1/10 mm, in accordance with ASTM Designation: D 5-71.[26]

   c. Volume concentration of the aggregate used in the mix defined by

$$C_v = \frac{\text{aggregate volume}}{\text{aggregate volume + bitumen volume}} \tag{F1}$$

2. The steps in using this method are as follows:

   a. With known values of penetration and ring-and-ball softening point, enter Figure F1 and determine the penetration index (PI).

   b. The next step involves the use of the nomograph presented in Figure F2. In addition to the PI, two other values are required: the temperature of the bituminous concrete mix for which the modulus value is desired, and the estimated loading frequency or time of loading that the prototype pavement will be subjected to. Use of a loading frequency of 2 Hz is recommended for taxiway design and 10 Hz for runway design. With values for the loading frequency and the difference in temperature between the bituminous concrete and the ring-and-ball softening point, a stiffness value for the bitumen S_{bit} can be determined from the appropriate PI line at the top of the nomograph. The value of S_{bit} is then used to determine the modulus of the mix S_{mix}.

   c. A value for S_{mix} may be determined by

$$S_{mix} = S_{bit} \left[1 + \left(\frac{2.5}{n}\right)\left(\frac{C_v}{1 - C_v}\right)\right]^n \tag{F2}$$

where

$$n = 0.83 \log \left(\frac{400{,}000}{S_{bit}}\right) \tag{F3}$$

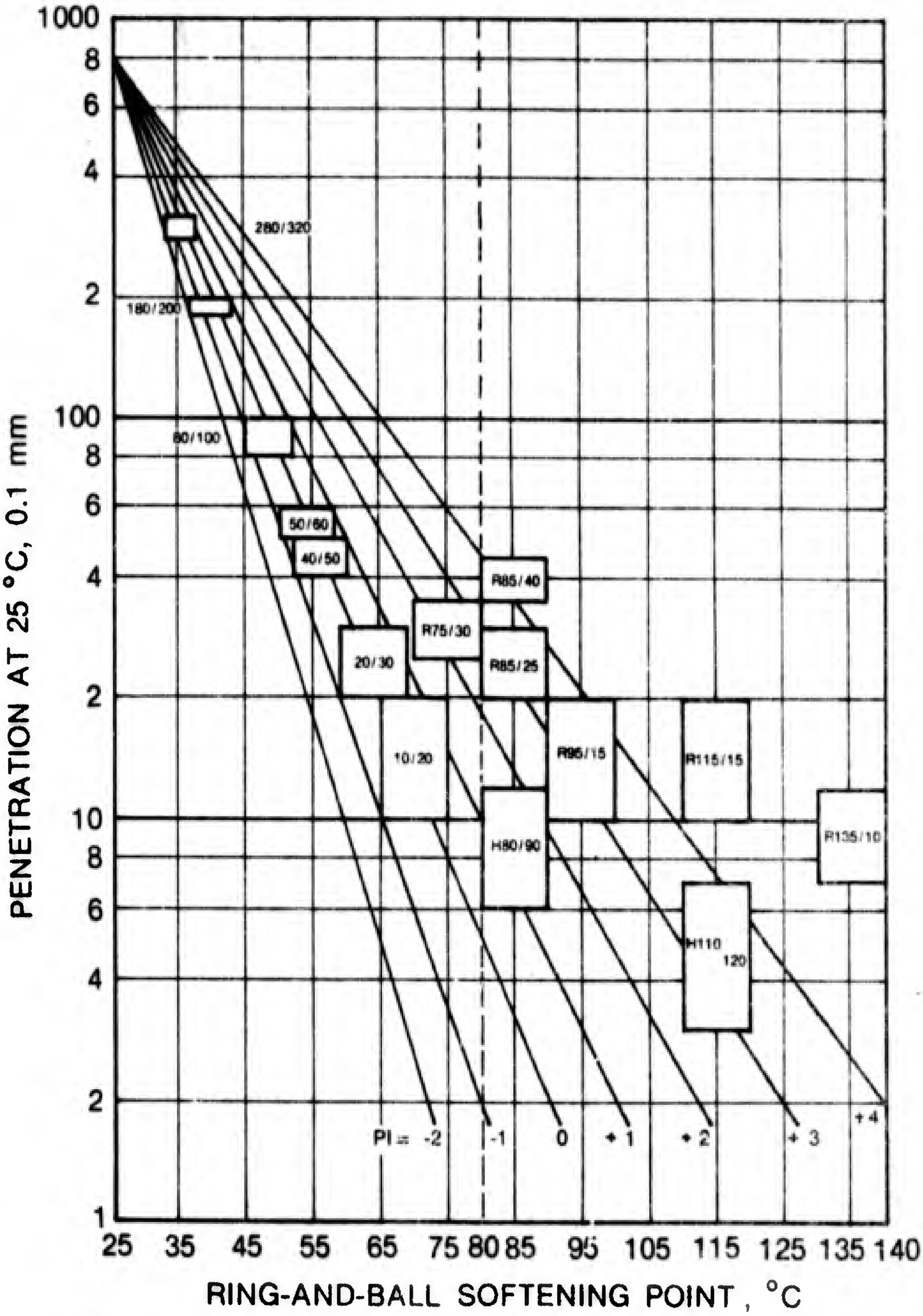


Figure F1. Relationship between penetration at 25° C and ring-and-ball softening point for bitumens with different PI's

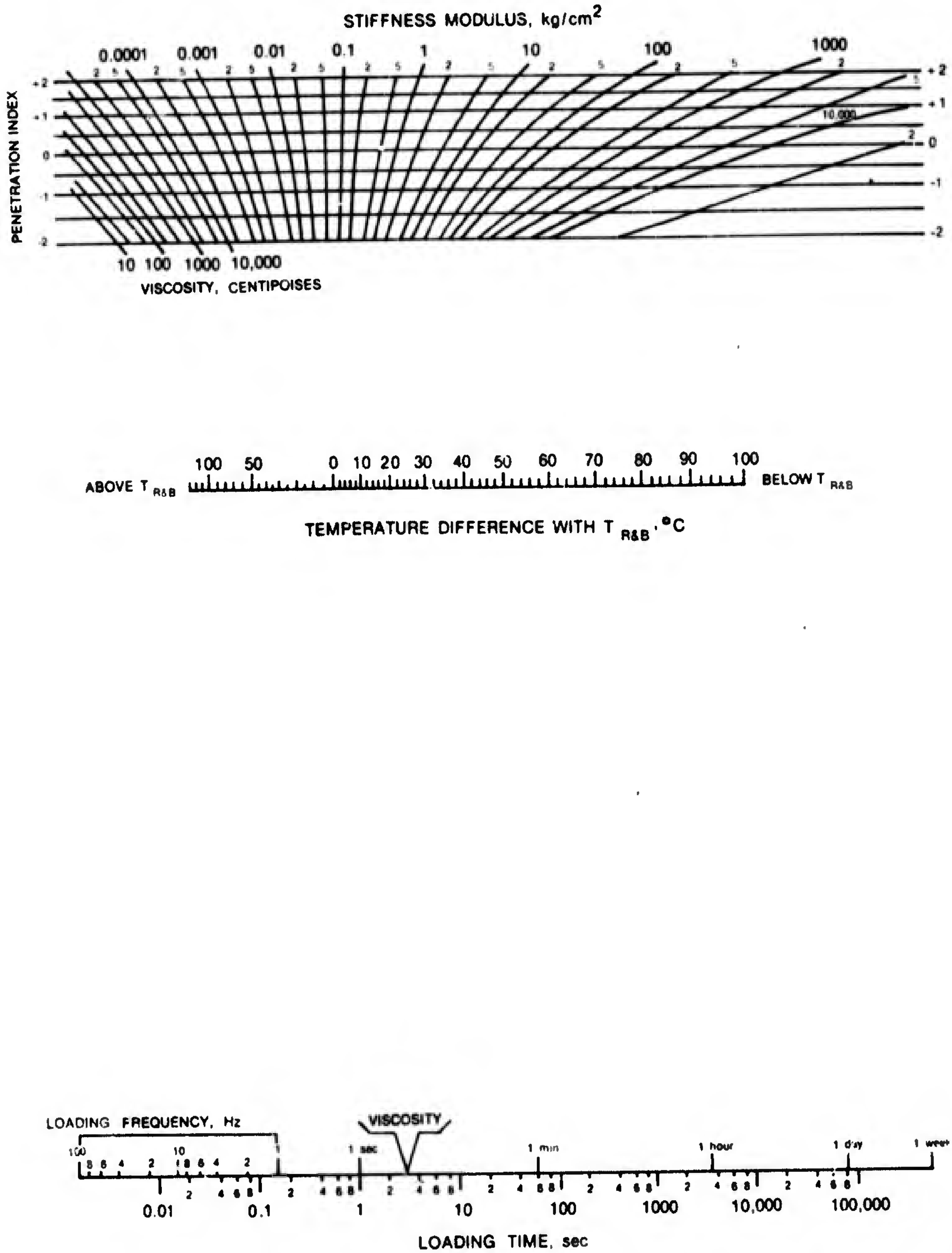


Figure F2. Nomograph for determining the stiffness modulus of bitumens (after Heukelom and Klomp[30])

The value thus determined for S_{mix} is in units of kilograms per square centimetre.

d. This expression should be used for aggregate volume concentrations of 0.7 to 0.9 and air void contents of 3 percent or less. For larger air void contents, use a corrected aggregate volume concentration

$$C'_v = \frac{C_v}{1 + \Delta\text{air void content}} \quad \text{(F4)}$$

where Δair void content is the actual air void content (expressed in decimal form) minus 0.03. Equation F4 is valid only when

$$C_B \geq \frac{2}{3}(1 - C'_v) \quad \text{(F5)}$$

where

$$C_B = \frac{\text{bitumen volume}}{\text{aggregate volume + bitumen volume}} \quad \text{(F6)}$$

APPENDIX G: PROCEDURE FOR DETERMINING THE MODULUS OF ELASTICITY OF UNBOUND GRANULAR BASE AND SUBBASE COURSE MATERIALS

1. The procedure presented in this appendix is based on relationships developed for the modulus of unbound granular layers as a function of the thickness of the layer and type of material.

2. The modulus relationships are shown in Figure G1. Modulus values for layer n (the upper layer) are indicated on the ordinate, and those for layer n + 1 (the lower layer) are indicated on the abscissa. Essentially linear relationships are indicated for various thicknesses of base and subbase course materials. For subbase courses, relationships are shown for thicknesses of 4, 5, 6, 7, and 8 in. For subbase courses having a design thickness of 8 in. or less, the applicable curve or appropriate interpolation can be used directly. For a design subbase course thickness in excess of 8 in., the layer should be divided into sublayers of approximately equal thickness and the modulus of each sublayer determined individually. For base courses, relationships are shown for thicknesses of 4, 6, and 10 in. These relationships can be used directly or by interpolation for design base course thicknesses up to 10 in. For design thicknesses in excess of 10 in., the layer should also be divided into sublayers of approximately equal thickness and the modulus of each sublayer determined individually.

3. To determine modulus values from this procedure, Figure G1 is entered along the abscissa using modulus values of the subgrade or underlying layer (modulus of layer n + 1). At the intersection of the curve applicable to this value with the appropriate thickness relationship, the value of the modulus of the overlying layer is read from the ordinate (modulus of layer n). This procedure is repeated using the modulus value just determined as the modulus of layer n + 1 to determine the modulus value of the next overlying layer.

4. For example, assume a pavement having a base course thickness of 4 in. and a subbase course thickness of 8 in. over a subgrade having

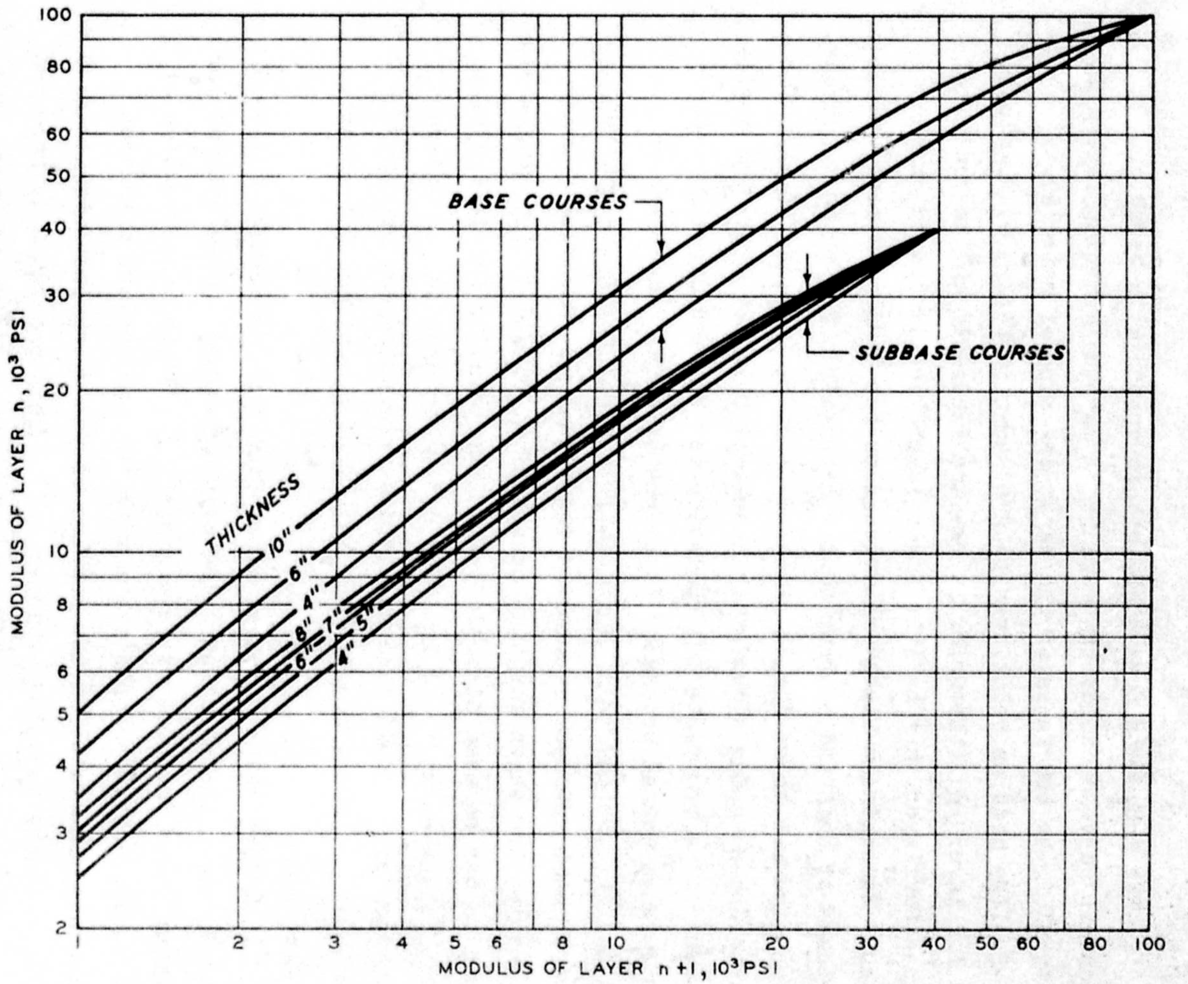


Figure G1. Relationships between modulus of layer n and modulus of layer n + 1 for various thicknesses of unbound base course and subbase course a modulus of 10,000 psi. Initially, the subgrade is assumed to be layer n + 1 and the subbase course to be layer n . Entering Figure G1 with a modulus of layer n + 1 of 10,000 psi and using the 8-in. subbase course curve, the modulus of the subbase (layer n) is found to be 18,500 psi. In order to determine the modulus value of the base course, the subbase course is now assumed to be layer n + 1 and the base course to be layer n . Entering Figure G1 with a modulus value of layer n + 1 of 18,500 psi and using the 4-in. base course relationship, the modulus of the base course is found to be 36,000 psi. Modulus values determined for each layer are indicated in Figure G2.

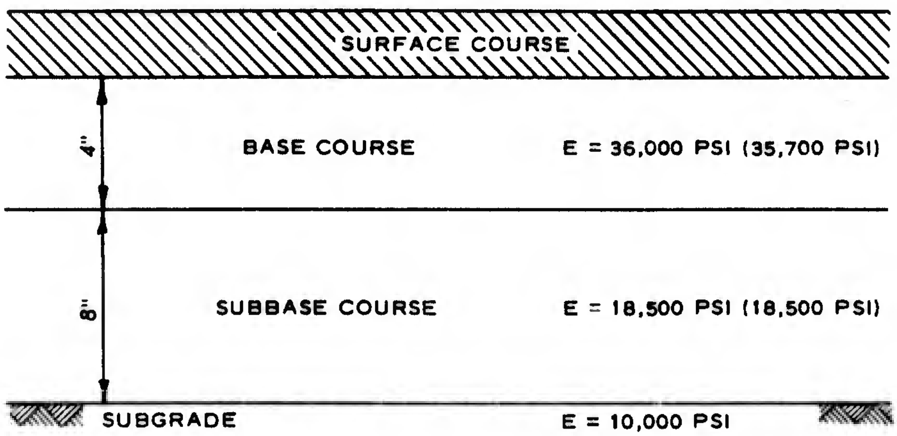


Figure G2. Modulus values determined for Example 1

5. If, in the above example, the design thickness of the subbase course had been 12 in., it would have been necessary to divide this layer into two 6-in.-thick sublayers. Then, using the procedure described above, the modulus values determined for the lower and upper sublayers of the subbase course and for the base course are 17,500, 25,500, and 44,000 psi, respectively. These values are shown in Figure G3.

6. The relationships indicated in Figure G1 can be expressed as

$$E_n = E_{n+1} (1 + 10.52 \log t - 2.10 \log E_{n+1} \log t)$$

for base course materials and as

$$E_n = E_{n+1} \left(1 + 7.18 \log t - 1.56 \log E_{n+1} \log t\right)$$

for subbase course materials. Use of these equations for direct computation of modulus values for the examples given above yields the values indicated in parentheses in Figures G2 and G3. It can be seen that comparable values are obtained with either graphical or computational determination of the modulus value for either material.

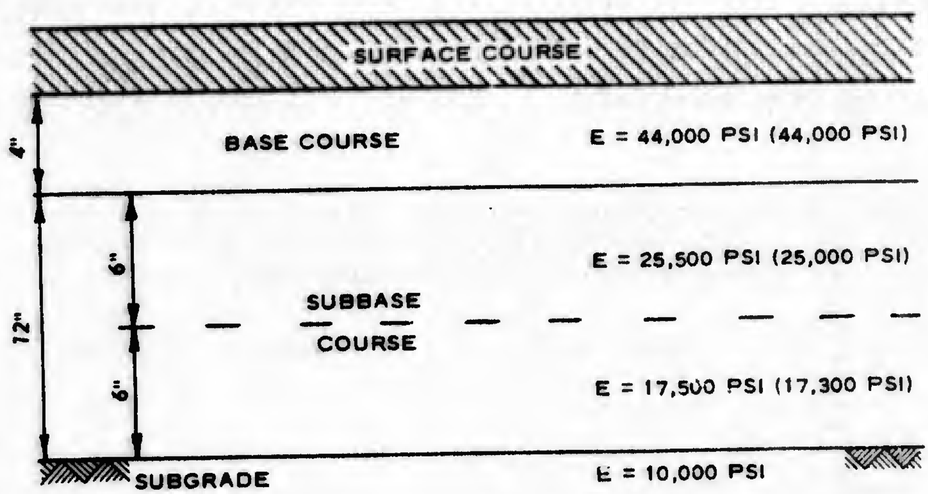


Figure G3. Modulus values determined for Example 2

APPENDIX H: PROCEDURES FOR DETERMINING THE FLEXURAL MODULUS AND FATIGUE CHARACTERISTICS OF CHEMICALLY STABILIZED SOILS

LABORATORY PROCEDURE

1. The procedure involves application of a repetitive loading to a laboratory prepared beam specimen under controlled stress conditions. Applied load and deflection along the neutral axis and at the lower surface are monitored, and the results are used to determine the flexural modulus and fatigue characteristics.

SPECIMEN PREPARATION

2. Beam specimens should be prepared following the general procedures indicated in American Society for Testing and Materials (ASTM) Designation: D 1632-63, "Making and Curing Soil-Cement Compression and Flexure Test Specimens in the Laboratory."[50] This method describes procedures for molding 3- by 3- by 11-1/4-in. specimens; however, any size mold may be used for the test. For soils containing aggregate particles larger than 3/4 in., it is recommended that molds on the order of 4 by 4 to 6 by 6 in. be used. In general, specimens should have an approximately square cross-sectional configuration and a length adequate to accommodate an effective test span equal to three times the height or width. Specimens should be molded to the stabilizer treatment level, moisture content, and density expected in the field structures. Cement-treated materials should be moist-cured for 7 days. Lime-treated materials should be cured for 28 days at 73° F.

EQUIPMENT

3. The following equipment is required:

   a. Loading frame capable of receiving specimen for third-point loading test.

   b. Electrohydraulic testing machine similar to that described in Appendix B. This machine must be capable of applying static and haversine loads.

   c. Load cell (approximately 2000-lb capacity).

   d. Two linear variable differential transformers (LVDT's) and one SR-4 type strain gage.

e. Recording equipment for monitoring deflection, strain, and load.

f. Miscellaneous pins and yokes, as described in the equipment setup below, for mounting the LVDT's.

EQUIPMENT SETUP

4. Details of the equipment setup are shown in Figures H1-H3. The beam should be positioned so that the molding laminations are horizontal. The three yokes are positioned over the top of the beam and held in place by threaded pins positioned along the neutral axis. The end pins, Pins A and C, are positioned directly over the end reaction points, and the middle pin, Pin B, is positioned at the center of the beam. A metal bar rests on top of the pins. At the A position, the bar is equipped with a lower vertical tab having a hole that slips loosely over the pin. A nut is placed on the end of the pin to prevent the bar from slipping. At the center or B position, the bar is equipped with a vertical tab onto which an LVDT is cemented in a vertical position. At this position on the bar, there is a hole through which the LVDT core pin falls to rest on the B pin. This pin must be fabricated with flat sides on the shaft to provide a horizontal surface on which the LVDT core pin rests. At the C position, the end of the bar simply rests on the unthreaded portion of the C pin. A nut is placed on the end of the C pin to prevent excessive side movement of the bar end. This type of bar, pin, and LVDT arrangement is provided on both sides of the beam. Although no dimensions are provided in Figures H1-H3, this type of equipment can easily be dimensioned and fabricated to fit any size beam. Either steel or aluminum may be used. The beam should be positioned and arranged to accommodate third-point loading as indicated in Figure H2. As the beam bends under loading, deflection at the center is measured by determining the movement of the LVDT stems from their original positions. The LVDT's are connected to the monitoring system to give an average deflection reading. Since it is also desired to determine the maximum tensile strain of the beam under loading, an SR-4 strain gage should be attached to the lower beam surface with epoxy or some other suitable cement and should also be connected to the

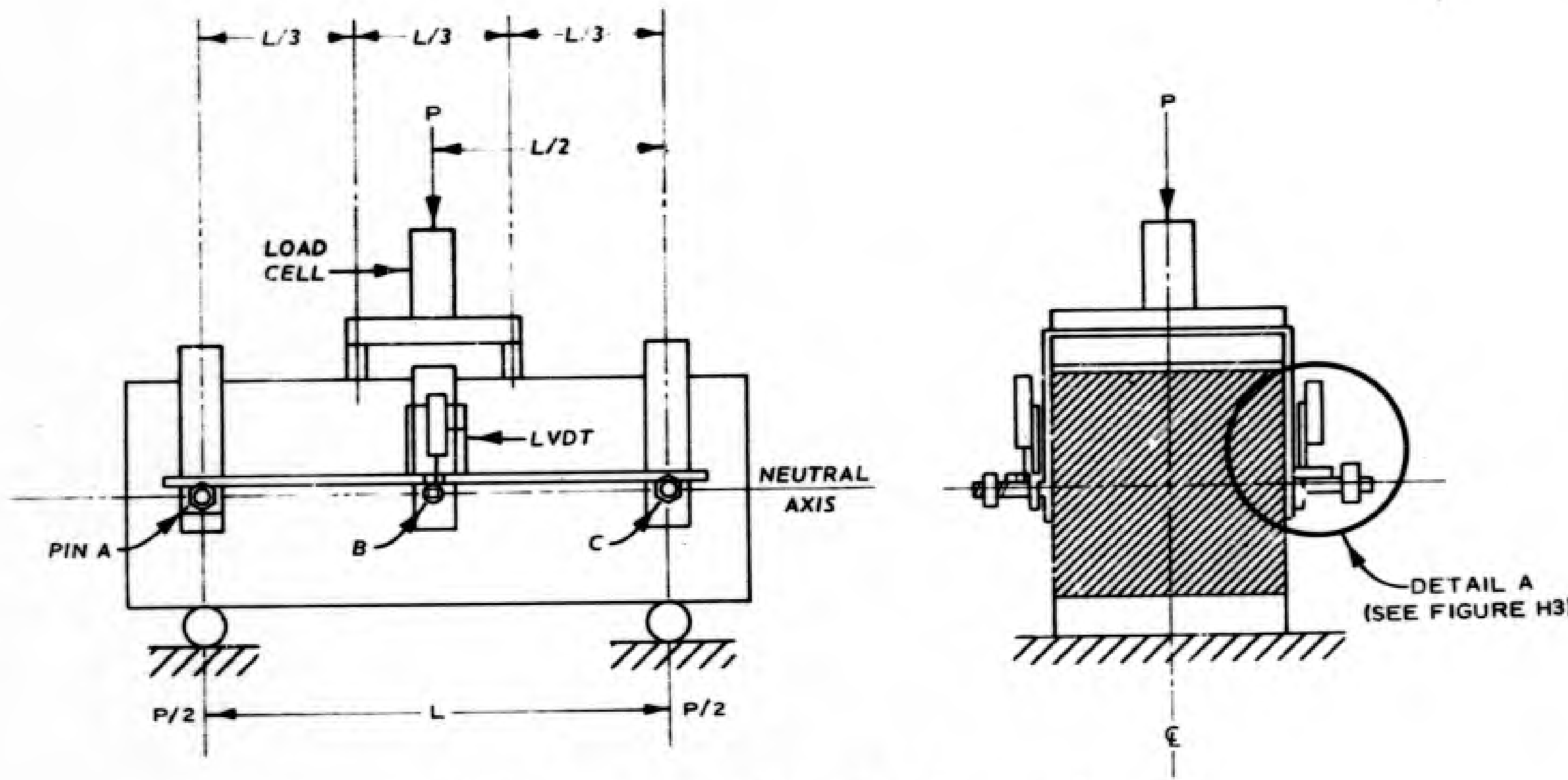


Figure H1. General view of equipment setup

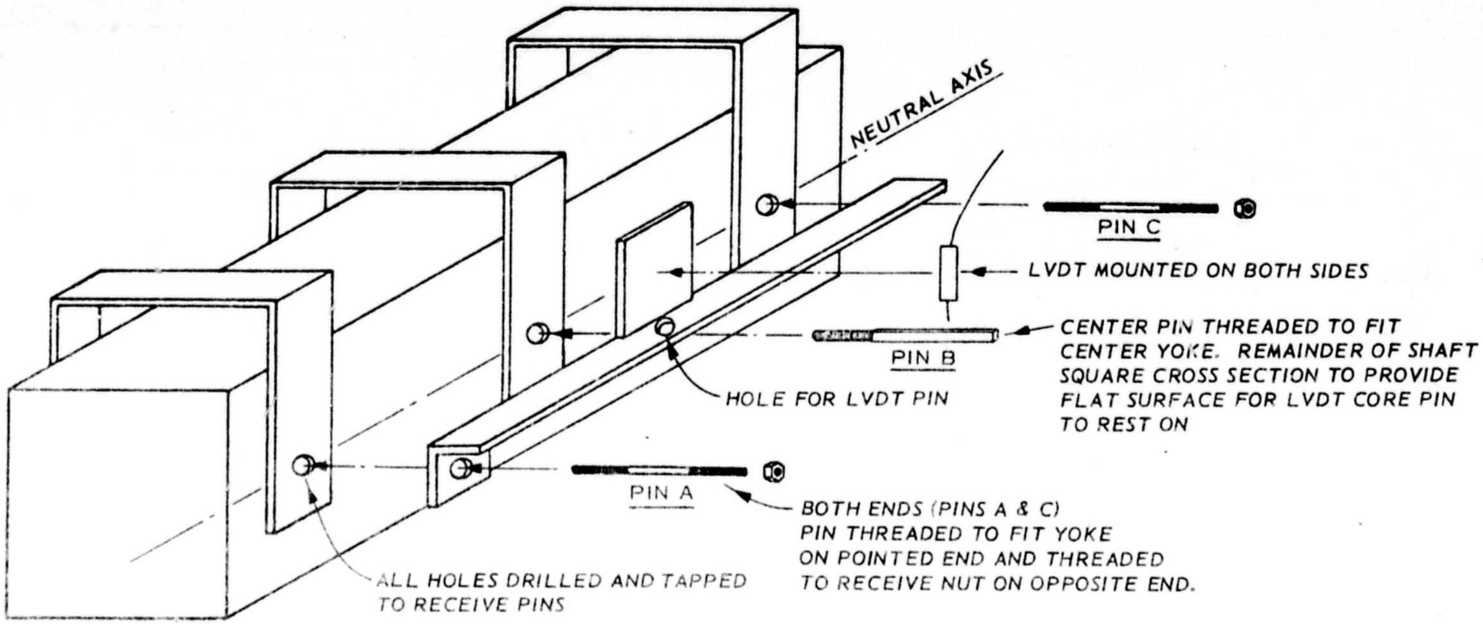


Figure H2. Details of equipment setup

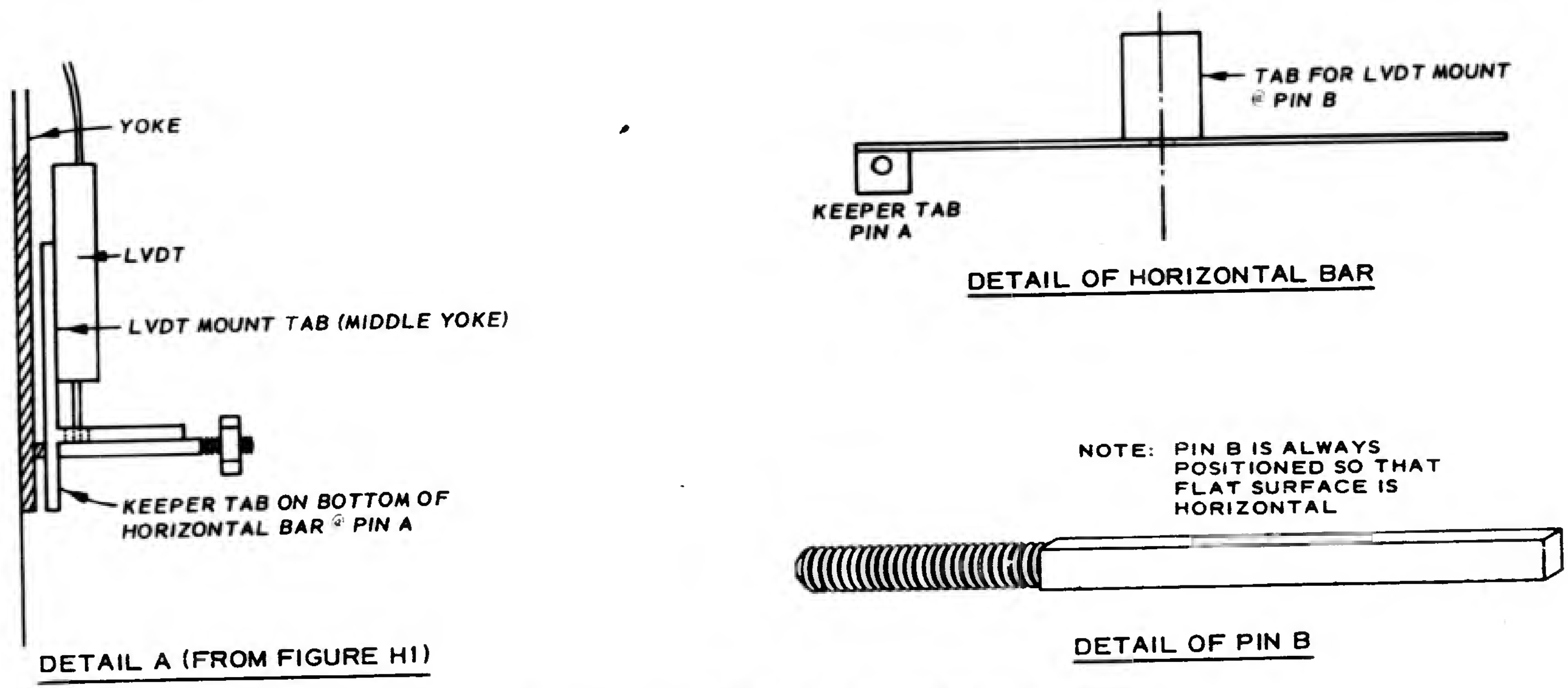


Figure H3. Miscellaneous details

monitoring system. If it is not possible to determine strain directly, a strain value may be found using the formula indicated hereafter.

TEST PROCEDURE

5. The flexural beam test is a stress-controlled test. Therefore, an initial specimen should be statically loaded to failure, and the stress level for the initial repetitive load tests should be set at 50 percent of the maximum rupture load. The repetitive load test should be conducted using a haversine wave form, a loading duration of 0.5 sec, and a frequency of about 1 Hz. To develop a strain repetition pattern, it is recommended that tests be conducted at 40 percent, 50 percent, 60 percent, and 70 percent of the maximum rupture value; however, stress levels can be varied to higher or lower levels. Data to be monitored include load, deflection along the neutral axis, strain at the lower surface of the specimen, and number of repetitions.

REPORTING OF TEST RESULTS

6. Flexural Modulus. The flexural modulus should be determined at 100, 1,000, and 10,000 load repetitions or at failure. This value may be determined from load and deflection data monitored at these repetition levels using the expression

$$E_f = \frac{23PL^3}{1296dI}\left[1 + 2.11\left(\frac{h}{L}\right)^2\right] \qquad \text{(H1)}$$

where

E_f = flexural modulus, psi
P = maximum load amplitude, lb
L = specimen length, in.
d = deflection at the neutral axis, in.
I = moment of inertia, in.^{4}
h = specimen height, in.

The value to be used for E_f in the performance model is the arithmetic mean of all values obtained during the test.

7. Fatigue Characteristics. Fatigue characteristics are presented as a plot of strain indicated at the bottom surface of the specimen versus load repetitions at failure. Generally the value of

the strain obtained during the first few load repetitions is the value to be plotted. If no direct means of measuring strain is available, a strain value ε may be computed using the expression

$$\varepsilon = \frac{PLh}{6E_f I} \tag{H2}$$

GRAPHICAL DETERMINATION OF FLEXURAL MODULUS FOR CHEMICALLY STABILIZED SOILS (CRACKED SECTION)

8. The procedure for determining a flexural modulus value for chemically stabilized soils based on the cracked section concept involves the use of a relationship between unconfined compressive strength and flexural modulus determined analytically from previous test results as discussed in the main text of this report. This relationship is shown in Figure 11 of the main text. To use this relationship, specimens of the stabilized material should be molded and tested following procedures indicated in ASTM Designation: D 1633-63, "Standard Method of Test for Compressive Strength of Molded Soil-Cement Cylinders."[50] Values obtained from the unconfined compression test can then be used to determine the values of the equivalent cracked section modulus using Figure 11.

APPENDIX I: PROCEDURES FOR DETERMINING THE FATIGUE LIFE OF BITUMINOUS CONCRETE

LABORATORY TEST METHOD

1. A laboratory procedure for determining the fatigue life of bituminous concrete paving mixtures containing aggregate with maximum sizes up to 1-1/2 in. is described in the first part of this appendix. The fatigue life of a simply supported beam specimen subjected to third-point loadings applied during controlled stress-mode flexural fatigue tests is determined.

DEFINITIONS

2. The following symbols are used in the description of this procedure:

<u>a</u>. ε = initial extreme fiber strain (tensile and compressive), in./in.

<u>b</u>. N_f = fatigue life of the specimen, number of load repetitions to fracture.

3. Extreme fiber strain of simply supported beam specimens subjected to third-point loadings, which produces uniaxial bending stresses, is calculated from

$$\varepsilon = \frac{12td}{(3L^2 - 4a^2)} \qquad \text{(I1)}$$

where

t = specimen depth, in.

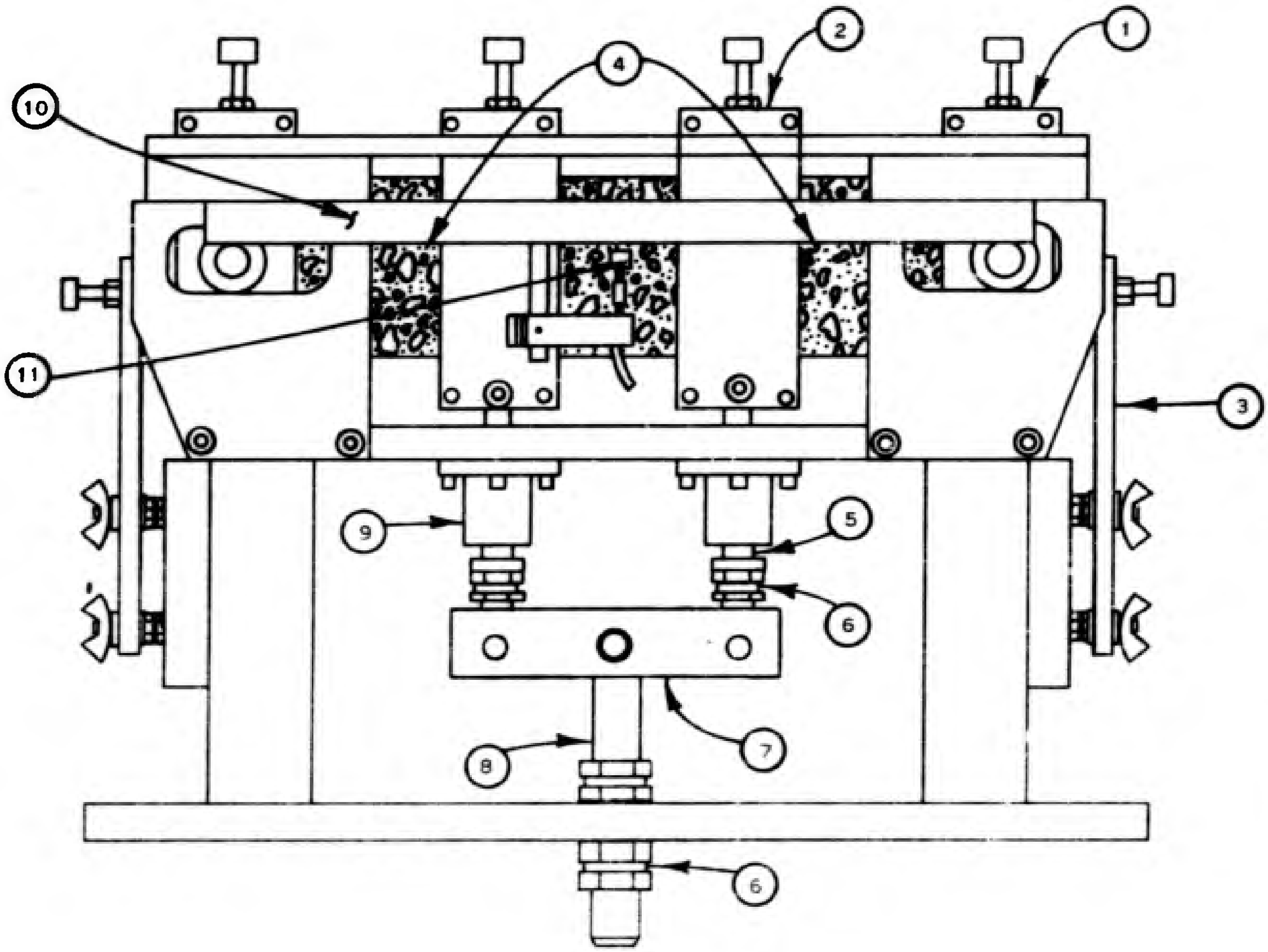
d = dynamic deflection of beam center, in.

L = reaction span length, in.

a = L/3, in.

TEST EQUIPMENT

4. The repeated flexure apparatus is shown in Figure I1. It accommodates beam specimens 15 in. long with widths and depths not exceeding 3 in. A 3000-lb-capacity electrohydraulic testing machine capable of applying repeated tension-compression loads in the form of haversine waves for 0.1-sec durations with 0.4-sec rest periods is used for flexural fatigue tests. Any dynamic testing machine or pneumatic



KEY

1. REACTION CLAMP
2. LOAD CLAMP
3. RESTRAINER
4. SPECIMEN
5. LOADING ROD
6. STOP NUTS
7. LOAD BAR
8. PISTON ROD
9. THOMPSON BALL BUSHING
10. LVDT HOLDER
11. LVDT

Figure I1. Repeated flexure apparatus

pressure system with similar loading capabilities is also suitable. Third-point loading, i.e., loads applied at distances of L/3 from the reaction points, produces an approximately constant bending moment over the center 4 in. of a 15-in.-long beam specimen with widths and depths not exceeding 3 in. A sufficient load, approximately 10 percent of the load deflecting the beam upward, is applied in the opposite direction forcing the beam to return to its original horizontal position and holding it at that position during the rest period. Adjustable stop nuts installed on the flexure apparatus loading rod prevent the beam from bending below the initial horizontal position during the rest period.

5. The dynamic deflection of the beam's center is measured with a linear variable differential transformer (LVDT). An LVDT that has been found suitable for this purpose is the Shaevitz type 100 M-L. The LVDT core is attached to a nut bonded with epoxy cement to the center of the specimen. Outputs of the LVDT and the electrohydraulic testing machine's load cell, through which loads are applied and controlled, can be fed to any suitable recorder. The repeated flexure apparatus is enclosed in a controlled-temperature cabinet capable of controlling temperatures within ± 1/2° F. A Missimer's model 100 x 500 CO_2 plug-in temperature conditioner has been found to provide suitable temperature control.

SPECIMEN PREPARATION

6. Beam specimens 15 in. long with 3-1/2 in. depths and 3-1/4-in. widths are prepared according to American Society for Testing and Materials (ASTM) Method D 3202.[26] If there is undue movement of the mixture under the compactor foot during beam compaction, the temperature, foot pressure, and number of tamping blows should be reduced. Similar modifications to compaction procedures should be made if specimens with less density are desired. A diamond-blade masonry saw is used to cut 3-in. or slightly less deep by 3-in. or slightly less wide test specimens from the 15-in.-long beams. Specimens with suitable dimensions can also be cut from pavement samples. The widths and depths of the specimens are

measured to the nearest 0.01 in. at the center and at 2 in. from both sides of the center. Mean values are determined and used for subsequent calculations.

TEST PROCEDURES

7. Repeated flexure apparatus loading clamps are adjusted to the same level as the reaction clamps. The specimen is clamped in the fixture using a jig to position the centers of the two loading clamps 2 in. from the beam center and to position the centers of the two reaction clamps 6-1/2 in. from the beam center. Double layers of Teflon sheets are placed between the specimen and the loading clamps to reduce friction and longitudinal restraint caused by the clamps.

8. After the beam has reached the desired test temperature, repeated loads are applied. Duration of a load repetition is 0.1 sec with 0.4 sec rest periods between loads. The applied load should be that which produces an extreme fiber stress level suitable for flexural fatigue tests. For fatigue tests on typical bituminous concrete paving mixtures, the following ranges of extreme fiber stress levels are suggested:

Temperature, °F	Stress Level Range, psi
55	150 to 450
70	75 to 300
85	35 to 200

The beam center point deflection and applied dynamic load are measured immediately after 200 load repetitions for calculation of extreme fiber strain ε . The test is continued at the constant stress level until the specimen fractures. The apparatus and procedures described have been found suitable for flexural fatigue tests at temperatures ranging from 40° to 100° F and for extreme fiber stress levels up to 450 psi. Extreme fiber stress levels for flexural fatigue tests at any temperature should not exceed that which causes specimen fracture before at least 1000 load repetitions are applied.

9. A set of 8 to 12 fatigue tests should be run for each temperature to adequately describe the relationship between extreme fiber strain and the number of load repetitions to fracture. The extreme

fiber stress should be varied such that the resulting number of load repetitions to fracture ranges from 1,000 to 1,000,000.

REPORT AND PRESENTATION OF RESULTS

10. The report of flexural fatigue test results should include the following:

a. Density of test specimens.

b. Number of load repetitions to fracture N_f .

c. Specimen temperature.

d. Extreme fiber stress σ .

The flexural fatigue relationship may be presented in the form of a plot as shown in Figure I2.

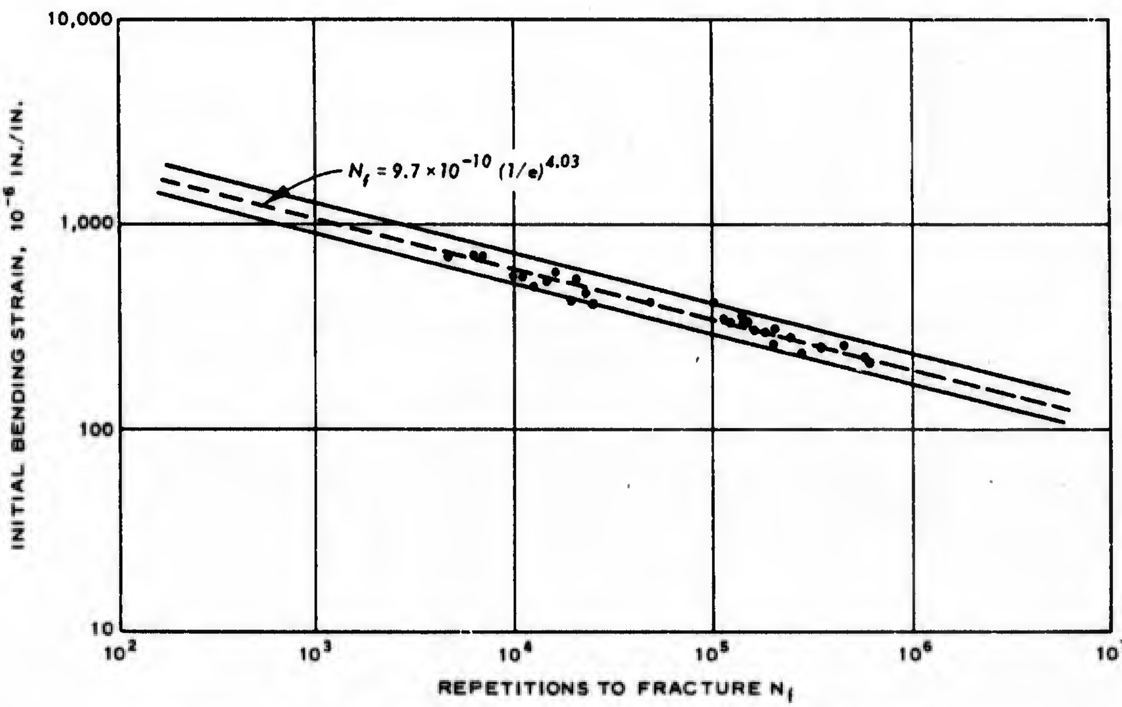


Figure I2. Initial mixture bending strain versus repetitions to fracture in controlled stress tests (after Pretorius[6])

PROVISIONAL FATIGUE DATA FOR BITUMINOUS CONCRETE

11. Use of the graph shown in Figure I3 to determine a limiting strain value for bituminous concrete involves first determining a value for the elastic modulus of the bituminous concrete. Using this value

and the design pavement service life in terms of load repetitions, the limiting tensile strain in the bituminous concrete can be read from the ordinate of the graph.

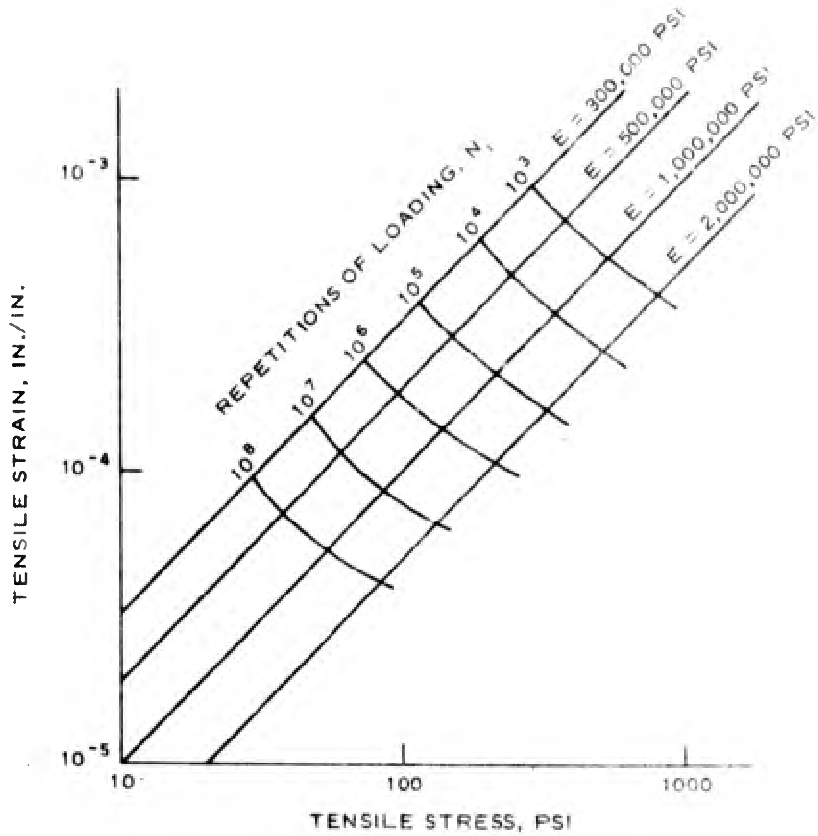


Figure I3. Provisional fatigue data for bituminous base course materials (after Heukelom and Klomp[30])

APPENDIX J: CHEVIT COMPUTER PROGRAM

1. The computer program used in the design procedure, CHEVIT, is a modified version of the CHEVRON computer program for computing deflections, strains, and stresses in a layered pavement system in which each layer is homogeneous, isotropic, and linear elastic. Although CHEVIT as it is now developed has the capability of performing iterative solutions for consideration of stress-dependent properties of materials, this feature of the program was not used in the development of the limiting strain criteria nor is it used in the design procedure. The modifications for computations of pavement response to mulitiple-wheel loadings consisted of minor changes to the CHEVRON program and the addition of two subroutines, WHEELS and FIT. The subroutine MOD shown in the program listing is for computing stress-dependent moduli.

COMPUTATIONAL PROCEDURE FOR COMPUTING PAVEMENT RESPONSE TO MULTIPLE-WHELL LOADINGS

2. For single-wheel loads, CHEVIT utilizes the basic CHEVRON program for computing pavement response parameters and no procedural changes are involved. If multiple-wheel loadings are specified, the subroutines WHEELS and FIT are also required for performing the necessary stress and strain rotations and superpositions. The basic procedure is to first use CHEVRON to establish a function for each given depth that relates each response parameter to the radial offset. To establish the function, the response parameters are computed at nine offset distances, 0.0, 0.5, 1.0, 1.5, 2.0, 3.0, 4.0, 8.0, and 12.0 radii, from the center of the load area. These values are stored and furnish the data by which subroutine FIT establishes a function for computing the pavement response parameter at any distance. After computing and storing the data for all the depths, the subroutine WHEELS is entered. In this subroutine, all of the data pertaining to the configuration of the multiple-wheel gear and the locations of the computational depths are read. The subroutine then computes the distances from each tire to the computational point, uses the stored data and subroutine FIT to compute the pavement response parameter at that point, rotates

stresses and strains to the gear coordinate system, and superimposes the response parameters for the different tires. The particular system for computing pavement response parameters for multiple-wheel gears was developed as an economical means for determining the response parameters at a large number of points for a given depth. The economy of the procedure over other procedures is realized when the number of computational points at a given depth times the number of tires in a gear is ten or greater and increases as the number of tires and/or computational points increases.

INPUT GUIDE

3. The following are general notes on the input guide:

a. All data are input in a floating point format of F10.0.

b. It should be noted that when a multiple-wheel gear is to be input it is necessary to input a zero for the number of radial offsets. This is necessary because the radial offsets are preset in the program for establishing the function relating the response parameters to offset distances. For this case, the input table for inputting the radial offsets is bypassed.

c. For a single-wheel gear problem, the response parameters may be computed for up to 99 different depths (each interface will be considered as 2 depths). For a multiple-wheel gear problem, the number of depths is limited to 15 (again, each interface is considered 2 depths). If the allowable number of depths is exceeded, the program will terminate.

d. The input guide is listed in Tables J1-J14, and each table represents a specific load statement. All problems must start with the word START in the first five columns of the first card. The word END in the first three columns terminates the run and is used to end the run after all problems have been completed.

Tables J1-J3

GENERAL PURPOSE DATA FORM

PROGRAM

REQUESTED BY | PREPARED BY | CHECKED BY | DATE | PAGE OF

Table 1: Option Card (A5 or A3)

Columns 1-5 THEEND = START, continue input

START

or Columns 1-3 THEEND = END, terminates job

END

Table 2: Identification Card (20A4)

Columns 1-80 TITLE = problem description

TITLE(1) - TITLE(20)

Table 3: Wheel Card (2F10.0)

Columns 1-10 WGT = wheel load, lb

Columns 11-20 PSI = tire contact pressure, psi

WGT	PSI

WES FORM NO. 1233
SEPTEMBER 1962

Tables J4 and J5

GENERAL PURPOSE DATA FORM

PROGRAM

REQUESTED BY | PREPARED BY | CHECKED BY | DATE | PAGE OF

Table 4: Layer Card (F10.0)

Columns 1-10 XNS = number of layers

1-10	11-20	21-30	31-40	41-50	51-60	61-70	71-80
XNS							

Table 5: Layer Properties Card (8F10.0)

Columns 1-10 E(1) = modulus of elasticity, layer 1

Columns 11-20 V(1) = Poisson's ratio, layer 1

.

.

.

Columns 71-80 V(4) = Poisson's ratio, layer 4 (if needed)

1-10	11-20	21-30	31-40	41-50	51-60	61-70	71-80
E(1)	V(1)	E(2)	V(2)	E(3)	V(3)	E(4)	V(4)

If XNS > 4 , continue to next data card using same format until XNS is satisfied

1-10	11-20	21-30	31-40	41-50	51-60	61-70	71-80
E(5)	V(5)	E(XNS)	V(XNS)				

WES FORM NO. 1233
SEPTEMBER 1962

Tables J6 and J7

GENERAL PURPOSE DATA FORM

PROGRAM | DATE

REQUESTED BY | PREPARED BY | CHECKED BY | PAGE OF

Table 6: Layer Thickness Card (8F10.0)

Columns 1-10 HH(1) = thickness of layer 1, in.

Columns 11-20 HH(2) = thickness of layer 2, in.

.

.

.

Columns 71-80 HH(8) = thickness of layer 8, in. (if needed)

1-10	11-20	21-30	31-40	41-50	51-60	61-70	71-80
HH(1)	HH(2)	HH(3)	HH(4)	HH(5)	HH(6)	HH(7)	HH(8)

If (XNS - 1) > 8 , continue to next data card using same format until (XNS - 1) is satisfied

1-10	11-20	21-30
HH(9)	HH(10)	HH(XNS-1)

Table 7: Offset Card (F10.0)

Columns 1-10 XIR = number of offsets

1-10
XIR

If XNW from Table 11 is > 1, set XIR = 0 and branch to Table 9

WES FORM NO. 1233
SEPTEMBER 1962

Tables J8 and J9

GENERAL PURPOSE DATA FORM

PROGRAM
REQUESTED BY | PREPARED BY | CHECKED BY | DATE | PAGE OF

Table 8: Offset Distance Card (8F10.0). Not applicable if XNW > 1 (multiple wheels)

Columns 1-10 RR(1) = distance to first offset, in.

Columns 11-20 RR(2) = distance to second offset, in.

.
.
.

Columns 71-80 RR(8) = distance to eighth offset, in. (if needed)

RR(1)	RR(2)	RR(3)	RR(4)	RR(5)	RR(6)	RR(7)	RR(8)

If XIR > 8, continue to next data card using same format until XIR is satisfied

RR(9)	RR(10)	RR(XIR)

Table 9: Computational Depths Card (F10.0)

Columns 1-10 XIZ = number of depths to computational points

XIZ

WES FORM NO. 1233
SEPTEMBER 1962

Table J10

GENERAL PURPOSE DATA FORM

PROGRAM

REQUESTED BY | PREPARED BY | CHECKED BY | DATE | PAGE OF

Table 10: Depths to Computational Points Card (8F10.0)

Columns 1-10 ZZ(1) = first computational depth, in.

Columns 11-20 ZZ(2) = second computational depth, in.

.

.

.

Columns 71-80 ZZ(8) = eighth computational depth, in.

1-10	11-20	21-30	31-40	41-50	51-60	61-70	71-80
ZZ(1)	ZZ(2)	ZZ(3)	ZZ(4)	ZZ(5)	ZZ(6)	ZZ(7)	ZZ(8)

If XIZ > 8 , continue to next data card using same format until XIZ is satisfied

1-10	11-20	21-30
ZZ(9)	ZZ(10)	ZZ(XIZ)

NOTE: The main program, CHEVIT, is dimensioned for 99 ZZ's. At each interface of ZZ, two computational points are generated, one in each layer. Subroutine WHEELS is dimensioned for 15 ZZ's only. Therefore, if WHEELS is utilized, the maximum number of ZZ's is 15. Remember, each interface depth ZZ(i) represents 2 ZZ's in storage.

(This ends the input into the main program. If WHEELS is not to be used, place a blank card after Table 10. Otherwise, proceed to Table 11.)

WES FORM NO. 1233
SEPTEMBER 1962

Tables J11 and J12

GENERAL PURPOSE DATA FORM

PROGRAM				DATE
REQUESTED BY		PREPARED BY	CHECKED BY	PAGE OF

Columns 1–10	Columns 11–20
XNW	
XW(1)	YW(1)
XW(2)	YW(2)
.	.
.	.
.	.
XW(XNW)	YW(XNW)

Table 11: Wheels Card (F10.0)

Columns 1-10 XNW = number of wheels in aircraft assembly

Table 12: Coordinates Card (2F10.0)

Columns 1-10 XW(1) = X-coordinate of first wheel, in.

Columns 11-20 YW(1) = Y-coordinate of first wheel, in.

(This format is repeated until XNW is satisfied.)

WES FORM NO. 1233
SEPTEMBER 1962

Tables J13 and J14

GENERAL PURPOSE DATA FORM

PROGRAM
REQUESTED BY | PREPARED BY | CHECKED BY | DATE | PAGE OF

Table 13: Grid Card (F10.0)

Columns 1-10 XNGRID = number of grid generator cards to follow

Columns 1-10
XNGRID

Table 14: Grid Generator Card (6F10.0)

Columns 1-10 XXP = initial X-coordinate of grid
Columns 11-20 YP = initial Y-coordinate of grid
Columns 21-30 DX = step size in X-direction
Columns 31-40 DY = step size in Y-direction
Columns 41-50 XNUMX = number of lines in X-direction
Columns 51-60 XNUMY = number of lines in Y-direction

1-10	11-20	21-30	31-40	41-50	51-60
XXP	YP	DX	DY	XNUMX	XNUMY

Repeat this card for each NGRID

(This completes the input for one problem. To begin another problem, simply return to Table 1 and use the START option. If no additional problems are desired, use the END option.)

WES FORM NO. 1233
SEPTEMBER 1962

PROGRAM LISTING

4. A complete listing of the computer program is presented on the following 25 pages.

```
C CHEVIT  -  N-LAYER ELASTIC SYSTEMS PROGRAM
C     CALCULATING STRESSES, STRAINS, AND DEFLECTIONS
C
      COMMON   /RMCOY/RR(99),   ZZ(99),    E(15),    V(15),     HH(14),
C191MN  ****** MAIN ROUTINE - N-LAYER ELASTIC SYSTEM ******
     1                 H(14),    AZ(396),  A(396,15),B(396,15),C(396,15),
     2                 D(396,15),AJ(396),  RJ1(396), RJ0(396), TITLE(20),
     3                 TEST(11), BZ(100),  X(15,4,4),SC(14),   FM(2,2),
     4                PH(14,4,4),Q,        Z,        AR,       NS,
     5                 N,        L,        ITN,      RSZ,      RSR,
     6                 ROM,      RMU,      SF,       CSZ,      CST,
     7                 CSR,      CTR,      COM,      CMU,      PSI,
     8                 NLINE,    NOUTP,    NTEST,    I,        ITN4,
     9                 K,        LC,       JT,       TZZ,      PR,
     A                 PA,       P,        EP,       TIP,      TIM,
     B                 T1,       T2,       T3,       T4,       T5,
     C                 T6,       T2P,      T2M,      WA,       BJ1,
     D                 BJ0,      ZF,       SZ1,      SZ2,      SG1,
     E                 SG2,      PH,       PH2,      VK2,      VKP2,
     F                 VK4,      VKP4,     VKK8,     RDT,      RDS
      DIMENSION S(5,9,15),DEPTH(15),LAYER(15)
      DIMENSION LY(15),KAR(15),KARTER(15,5),IZZ(15)
      DIMENSION XLY(15),XKAR(15),XIZZ(15),XKARTER(15,5)
      DIMENSION  ZP(99)
      CHARACTER  END*3,THEEND*3
      REAL KARTER
      DATA END/3HEND/
      DATA   ASTER, PERD/4H****, 4H..../
C THIS CALL TO FXOPT SUPPRESSES AN OVERFLOW MESSAGE THAT RESULTS FROM
C CERTAIN COMPUTATIONS. ANOTHER COMPUTER MAY NOT HAVE THIS UNDERFLOW.
C THE UNDERFLOW IS NOT A SERIOUS CONDITION FOR THE TYPE RESULTS GENERATED.
      CALL FXOPT(67,1,1,0)
      NPUN=0
      NFILE = 0
      Y = 0.
      CALL FLGEOF (41,NFILE)
C
C         ** COMPUTE ZEROS OF J1(X) AND J0(X). SET UP GAUSS CONSTANTS **
      K = ITN+1
      DO 2 I=7,K,2
          T = I/2
          TD = 4.0*T - 1.0
    2     BZ(I) = 3.1415927*(T + 0.25 + 0.050661/TD
     1            -0.053041/TD**3 + 0.262051/TD**5)
      DO 3 I=8,ITN,2
          T = (I-2)/2
          TD = 4.0*T + 1.0
    3     BZ(I) = 3.1415927*(T + 0.25 - 0.151982/TD
     1            + 0.015399/TD**3 - 0.245270/TD**5)
C
   10 CONTINUE
C TABLE 1
```

```
      READ(5,390) THEEND
  390 FORMAT (A3)
      IF(THEEND.EQ.END) CALL EXIT
C TABLE 2
      READ(5, 310) TITLE                                    AUST1/74
      IF(NFILE.EQ.1) GO TO 9999
  310 FORMAT (20A4)
C       XNLIN IS CODE FOR NON-LINEAR PROBLEM, IF XNLIN=0
C       PROBLEM IS LINEAR AND NO NON-LINEAR PROPERTIES ARE
C       READ - IF XNLIN GREATER THAN 0 THEN XNLIN WILL BE THE
C       MAXIMUM CYCLES TO OBTAIN MODULUS -
C       ----
C       TOL IS TOLERANCE IN PERCENT FOR CLOSURE
C       ----
C TABLE 3
      READ(5,  1)WGT,PSI,XNLIN,TOL
  391 FORMAT(4F10.0)
      NLIN=XNLIN
      AR = SQRT (WGT/(3.14159*PSI))
  311 FORMAT(3F10.0)                                        AUST1/74
C TABLE 4
      READ(5,  1) XNS
      NS=XNS
  302 FORMAT(F10.0)                                         AUST1/74
C TABLE 5
      READ(5,  1) (E(I),V(I),I=1,NS)
  302 FORMAT(8F10.0)
    1 FORMAT(V)
      N = NS - 1
C TABLE 6
      READ(5,  1) (HH(I),I=1,N)
  313 FORMAT(6F10.0)                                        AUST1/74
C     IF NONLINEAR PROPERTIES TO BE USED GO TO STATEMENT 4000 .
      IF(NLIN.NE.0) GO TO 4000
 8031 CONTINUE
C TABLE 7
      READ(5,  1) XIR
      IR=XIR
      IF(IR.EQ.0) GO TO 8040
C TABLE 8
      READ(5,  1) (RR(I),I=1,IR)
 8040 CONTINUE
C TABLE 9
      READ(5,  1) XIZ
      IZ=XIZ
C TABLE 10
      READ(5,  1) (ZZ(I),I=1,IZ)
      IF(IR.NE.0)GO TO 3003                                 AUST1/74
      IR=9                                                  AUST1/74
      RADII=AR
      RR(1) = 0.0 * RADII
      RR(2) = 0.5 *RADII
```

```
      RR(3) = 1.0 * RADII
      RR(4) = 1.5 * RADII
      RR(5) = 2.0 * RADII
      RR(6) = 3.0 * RADII
      RR(7) = 4.0 * RADII
      RR(8) = 8.0 * RADII
      RR(9) = 12.0 * RADII                                         AUST1/74
 3003 CONTINUE
C NONLINEAR PART.
 4000 CONTINUE
      NLINE = 17+NS
      NPAGE = 1
      WRITE(6, 350) (ASTER,I=1,5),(TITLE(I),I=1,20),(ASTER,I=1,5),NPAGE
  350 FORMAT (1H1//1H0,5A4,1X,20A4,1X,5A4,6H  PAGE,I3)
      WRITE(6, 351) WGT,PSI,AR,(I,E(I),V(I),HH(I),I=1,N)
  351 FORMAT(1H0, 40X, 26HTHE PROBLEM PARAMETERS ARE/
     1      1H0, 20X, 12HTOTAL LOAD..., 8X, F10.2, 5H  LBS/
     2      1H0, 20X, 15HTIRE PRESSURE..., 5X, F10.2, 5H  PSI/
     3      1H0, 20X, 13HLOAD RADIUS..., 7X, F10.2, 5H  IN./ 1H /
     4      (1H , 20X, 5HLAYER, I3, 14H  HAS MODULUS ,F10.0,
     6       18H   POISSONS RATIO , F5.3, 17H   AND THICKNESS , F6.2,
     7       4H IN.))
      WRITE(6, 354) NS,E(NS),V(NS)
  354 FORMAT (1H , 20X, 5HLAYER, I3, 14H  HAS MODULUS ,F10.0,
     1       18H   POISSONS RATIO , F5.3, 24H   AND IS SEMI-INFINITE. )
  352 FORMAT (4X,1HR,5X,1HZ,12X,8HVERTICAL,4X,10HTANGENT   ,4X,6HRADIAL
     1,6X,5HSHEAR, 6X,4HBULK,7X,8HOCTSHEAR,2X,9HOCTSTRESS,4X,5HRATIO,
     1   2X,8HDEVIATOR  /)
      IF(NLIN.EQ.0)GO TO 4020
      ITRIP=0
C     INPUT NONLINEAR MATERIAL CHARACTERISTICS
      WRITE(6,4002)
 4002 FORMAT(5X,28HDATA FOR NON-LINEAR ANALYSIS)
C TABLE 6.1
      READ(5,  1)XIR
      IR=XIR
C TABLE 6.2
      READ(5,  1)(RR(I),I=1,IR)
C TABLE 6.3
      READ(5,  1)XIZ
      IZ=XIZ
      WRITE(6,4003)IR,TOL,NLIN,IZ
 4003 FORMAT(10X,26HNO. OF RADII TO USE--------,I5/
     810X,26HTOLERANCE FOR CLOSURE-----,F5.1,11H IN PERCENT/
     910X,26HMAXIMUM NUMBER OF CYCLES--,I5/
     110X,26HNO. OF NON-LINEAR LAYERS--,I5//6X,
     25HLAYER,6X,4HCODE,3X,5HDRTHS,3X,46H--------CHARACTERIZATION CONS
     3TANTS----------,/)
      K=0
C     READ    LAYER NUMBER CHARACTERIZATION CODE.DEPTH TO CHAR. LINE
C                  AND CHARACTERIZATION PARAMETERS
      DO 4010 I=1,IZ
```

```
C TABLE 4.4
      READ(5,  1)XLY(I),XKAR(I),XIZZ(I),(XKARTER(I,L),L=1,5)
      LY(I)=XLY(I)
      KAR(I)=XKAR(I)
      IZZ(I)=XIZZ(I)
      KARTER(I,1)=XKARTER(I,1)
      KARTER(I,2)=XKARTER(I,2)
      KARTER(I,3)=XKARTER(I,3)
      KARTER(I,4)=XKARTER(I,4)
      KARTER(I,5)=XKARTER(I,5)
      WRITE(6,4001)LY(I),KAR(I),IZZ(I),(KARTER(I,L),L=1,5)
      K=K+IZZ(I)
 4001 FORMAT(3I10,5F10.3)
 4010 CONTINUE
      IX=I2
      IZ=K
C TABLE 4.5
      READ(5,  1)(ZZ(I),I=1,IZ)
      WRITE(6,4007)
      K=1
      DO 4011 I=1,IX
      J=K+IZZ(I)-1
      WRITE(6,4008)LY(I),(ZZ(L),L=K,J)
 4011 K=J+1
 4007 FORMAT(//6X,5HLAYER,20X,
     140H-------DEPTHS TO REFERENCE POINTS------/)
 4008 FORMAT(I10,(7F10.2))
      NPAGE=NPAGE+1
      WRITE(6,4004) NPAGE
 4004 FORMAT(1H1,30X,25H---NON-LINEAR ANALYSIS---,33X,19H**************
     1*****,15X,6H PAGE,I3//)
C  ** ADJUST LAYER DEPTHS **
 4020 CONTINUE
      ITRIP=ITRIP+1
      IF(NLIN.NE.0 .AND. NPAGE.GT.2)WRITE(6,4050)
      IF(NLIN.NE.0 .AND. NPAGE.GT.2)WRITE(6,4051)ITRIP,NPAGE
      IF(NLIN.NE.0 .AND. NPAGE.EQ.2)WRITE(6,4005)ITRIP
      IF(NLIN.NE.0 .AND. NPAGE.EQ.2)NLINE=10
      IF(NLIN.NE.0 .AND. NPAGE.GT.2)NLINE=6
 4050 FORMAT(1H1)
 4005 FORMAT(/63H---STRESSES FOR DETERMINATION OF MODULUS VALUES FOR CYC
     1LE NO---,I5//)
 4051 FORMAT(/63H---STRESSES FOR DETERMINATION OF MODULUS VALUES FOR CYC
     1LE NO---,I5,54X,6H PAGE,I3//)
      WRITE(6,359)
  359 FORMAT(//)
      WRITE (6,352)
      H(1)=HH(1)
      DO 25 I=2,N
   25 H(I)=H(I-1)+HH(I)
      IRT=0
C  ** START ON A NEW R **
```

```
  100 IRT=IRT+1
      IF(IRT-IR) 105,105,300                                      970
  105 R=RR(IRT)
      DO 31 I =1,IZ
      DO 31 J=1,N
      TZ  = ABS (H(J) - ZZ(I))
      IF(TZ - .0001) 32,32,31
   32 ZZ(I) = -H(J)
   31 CONTINUE
      WRITE(6, 355)
      NLINE = NLINE+1
  355 FORMAT(1H )
C  ** CALCULATE THE PARTITION **
      CALL PART
C  ** CALCULATE THE COEFFICIENTS **
      DO 125 I=1,ITN4
      P=AZ(I)
  107 CONTINUE
      IF(NS.GT.5) GOTO 108
      CALL COE5(I)
      GO TO 109
  108 CONTINUE
      CALL CO15(I)
  109   IF (R) 115,115,110
  110   PR = P*R
      CALL BESSEL (0,PR,Y)
          RJ0(I) = Y
      CALL BESSEL (1,PR,Y)
          RJ1(I) = Y
  115 PA=P*AR
      CALL BESSEL (1,PA,Y)
      AJ(I)=Y
  125 CONTINUE
  195 IZT=0
C  ** START ON A NEW Z **
          NDEP = 0                                                1315
  200 IZT=IZT+1
      IF(IZT-IZ) 205,205,100
  205 Z=ABS (ZZ(IZT))
          NDEP = NDEP + 1                                         1325
      ZP(NDEP)=ZZ(IZT)
      IF(NLINE-54)207,206,206
  206 NPAGE=NPAGE+1
      IF(NLIN-0)217,217,218
  218 NLINE=6
      WRITE(6,4050)
      WRITE(6,4051)ITRIP,NPAGE
      GO TO 207
  217 NLINE=8
      WRITE(6, 350) (ASTER,I=1,5),(TITLE(I),I=1,20),(ASTER,I=1,5),NPAGE
      WRITE (6,352)
  207 CONTINUE
```

```
C   ** FIND THE LAYER CONTAINING Z **
      IZZ = 0.0
      DO 210 J1=1,N
      J=NS-J1
      IF(Z-H(J)) 210,215,215
  210 CONTINUE
      L = 1
      GO TO 34
  215 L=J+1
      IF (ZZ(IZT)) 33,34,34
   33 L = J
      IZZ = 1.0
   34 CONTINUE
      CALL CALCIN
      IF (IZZ) 36,36,35
   35 ZZ(IZT) = -ZZ(IZT)
      IZT = IZT-1
   36 CONTINUE
         DEPTH(NDEP) = Z                                       1581
         S (1,IRT,NDEP) = CSZ                                  1582
         S (2,IRT,NDEP) = CST                                  1583
         S (3,IRT,NDEP) = CSR                                  1584
         S (4,IRT,NDEP) = CTR                                  1585
         S (5,IRT,NDEP) = COM                                  1586
         LAYER(NDEP) = L                                       1587
      GO TO 200
  300 CONTINUE                                                 1591
      IF(NLIN.EQ.0) GO TO 303
      IF(NLINE-54)4061,4062,4062
 4062 NLINE=8*IZ
      NPAGE=NPAGE+1
      WRITE(6,4065)NPAGE
 4065 FORMAT(1H1,122X,6H  PAGE,I3//)
 4061 CONTINUE
      CALL MOD(S,E,LY,KAR,KARTER,IX,IR,ZZ,TOL,NLIN,ITRIP,IZZ)
      IF(NLIN.EQ.0) GO TO 8031
      NPAGE=NPAGE+1
      GO TO 4020
  303 CONTINUE
 9002 FORMAT (F10.0,2F10.3)
      HH(NS) = 9999.9
      NPAGE=NPAGE+1
      WRITE(6, 350) (ASTER,I=1,5),(TITLE(I),I=1,20),(ASTER,I=1,5),NPAGE
      WRITE(6, 351) WGT,PSI,AR,(I,E(I),V(I),HH(I),I=1,N)
      WRITE(6, 354) NS,E(NS),V(NS)
      IF(NDEP.LT.16) GO TO 4040
      WRITE(6,9004)
 9004 FORMAT(//'NDEP,THE NO. OF COMPUTATIONAL DEPTHS, IS EXCEEDED....')
      STOP
 4040 CONTINUE
      CALL WHEELS(RR,NDEP,DEPTH,S,LAYER,E,V,AR,NS,PSI,PP,NPAGE)
      GO TO 10                                                 1593
```

```
 9999 CALL EXIT
      END
C
      BLOCK DATA
C
      COMMON   /RMCOY/RR(99),   ZZ(99),    E(15),      V(15),      HH(14),
     1             H(14),       AZ(396),   A(396,15), B(396,15), C(396,15),
     2             D(396,15), AJ(396),   RJ1(396),  RJ0(396),  TITLE(20),
     3             TEST(11),  BZ(100),   X(15,4,4), SC(14),    FM(2,2),
     4            PH(14,4,4), R,          Z,          AR,         NS,
     5             N,          L,          ITN,        RSZ,        RSR,
     6             ROM,        RMU,        SF,         CSZ,        CST,
     7             CSR,        CTR,        COM,        CMU,        PSI,
     8             NLINE,      NOUTP,      NTEST,      I,          ITN4,
     9             K,          LC,         JT,         TZP,        PR,
     A             PA,         P,          EP,         TIP,        TIM,
     B             T1,         T2,         T3,         T4,         T5,
     C             T6,         T2P,        T2M,        WA,         BJ1,
     D             BJ0,        ZF,         SZ1,        SZ2,        SG1,
     E             SG2,        PH,         PH2,        VK2,        VKP2,
     F             VK4,        VKP4,       VKK8,       RDT,        RDS
      DIMENSION ZB(6)
      EQUIVALENCE (BZ,ZB )
      DATA ZB/0.0,1.0,2.4048,3.8317,5.5201,7.0156/
      DATA ITN/46/,ITN4/184/
      END
C
      SUBROUTINE C015(KIN)
C       USED FOR PROBLEMS WITH 6 TO 15 LAYERS ONLY
C       ******SUBROUTINE COEE - N-LAYER ELASTIC SYSTEM ******
C
      COMMON   /RMCOY/RR(99),   ZZ(99),    E(15),      V(15),      HH(14),
     1             H(14),       AZ(396),   A(396,15), B(396,15), C(396,15),
     2             D(396,15), AJ(396),   RJ1(396),  RJ0(396),  TITLE(20),
     3             TEST(11),  BZ(100),   X(15,4,4), SC(14),    FM(4),
     4            PH(14,4,4), R,          Z,          AR,         NS,
     5             N,          L,          ITN,        RSZ,        RSR,
     6             ROM,        RMU,        SF,         CSZ,        CST,
     7             CSR,        CTR,        COM,        CMU,        PSI,
     8             NLINE,      NOUTP,      NTEST,      I,          ITN4,
     9             K,          LC,         JT,         TZP,        PR,
     A             PA,         P,          EP,         TIP,        TIM,
     B             T1,         T2,         T3,         T4,         T5,
     C             T6,         T2P,        T2M,        WA,         BJ1,
     D             BJ0,        ZF,         SZ1,        SZ2,        SG1,
     E             SG2,        PH,         PH2,        VK2,        VKP2,
     F             VK4,        VKP4,       VKK8,       RDT,        RDS
      LC = KIN
CS-MX       SET UP MATRIX X =DI*MI*KI*K*M*D
C     COMPUTE THE MATRICES X(K)
      IF (LC-1) 3,3,2
    3 ISW = 1
```

```
    2 IF(ISW) 99,1,1
    1 DO 10 K=1,N
      T1=E(K)*(1.0+V(K+1))/(E(K+1)*(1.0+V(K)))
      T1M=T1-1.0
      PH=P*H(K)
      PH2=PH*2.0
      VK2=2.0*V(K)
      VKP2=2.0*V(K+1)
      VK4=2.0*VK2
      VKP4=2.0*VKP2
      VKK8=8.0*V(K)*V(K+1)
C
      X(K,1,1)=VK4-3.0-T1
      X(K,2,1)=0.0
      X(K,3,1)=T1M*(PH2-VK4+1.0)
      X(K,4,1)=-2.0*T1M*P
C
      T3=PH2*(VK2-1.0)
      T4=VKK8+1.0-3.0*VKP2
      T5=PH2*(VKP2-1.0)
      T6=VKK8+1.0-3.0*VK2
C
      X(K,1,2)=(T3+T4-T1*(T5+T6))/P
      X(K,2,2)=T1*(VKP4-3.0)-1.0
      X(K,4,2)=T1M*(1.0-PH2-VKP4)
C
      X(K,3,4)=(T3-T4-T1*(T5-T6))/P
C
      T3=PH2*PH-VKK8+1.0
      T4=PH2*(VK2-VKP2)
C
      X(K,1,4)=(T3+T4+VKP2-T1*(T3+T4+VK2))/P
      X(K,3,2)=   (-T3+T4-VKP2+T1*(T3-T4+VK2))/P
C
      X(K,1,3)=T1M*(1.0-PH2-VK4)
      X(K,2,3)=2.0*T1M*P
      X(K,3,3)=VK4-3.0-T1
      X(K,4,3)=0.0
C
      X(K,2,4)=T1M*(PH2-VKP4+1.0)
      X(K,4,4)=T1*(VKP4-3.0)-1.0
   10 CONTINUE
C     COMPUTE THE PRODUCT MATRICES PM
      SC(N)=4.0*(V(N)-1.0)
      IF (N-2) 13,11,11
   11 DO 12 K1=2,N
      M=NS-K1
      SC(M)=SC(M+1)*4.0*(V(M)-1.0)
   12 CONTINUE
   13 CONTINUE
C
      DO 26 K1=1,N
```

```
      K = NS-K1
      IF (K-N) 23,21,21
   21 DO 22 M = 1,4
      DO 22 J = 1,4
   22 PM(K,M,J) = X(K,M,J)
      GO TO 26
   23 T1 = EXP (P*(H(K)-H(K+1)))
      T2 = 1.0/T1
      DO 20 M = 1,4
      DO 20 J = 1,4
   20 PM(K,M,J) = 0.0
      DO 25 M = 1,4
      DO 25 J = 1,4
         T6            = (X(K,M,1)*PM(K+1,1,J)+X(K,M,2)*PM(K+1,2,J))*T1 +
     C                   (X(K,M,3)*PM(K+1,3,J)+X(K,M,4)*PM(K+1,4,J))*T2
      IF (ABS (T6)-1.0E35) 24,24,99
   24 PM(K,M,J) = T6
   25 CONTINUE
   26 CONTINUE
      DO 30 K1=1,N
         T1 = EXP (-P*(H(N)+H(K1)))
         T2 = EXP (-P*(H(N)-H(K1)))
      DO 31 M=1,2
      DO 31 J=3,4
        PM(K1,M,J) = T1*PM(K1,M,J)
        PM(K1,M+2,J) = T2*PM(K1,M+2,J)
   31 CONTINUE
   30 CONTINUE
C     SOLVE FOR C(NS) AND D(NS)

      T3=2.0*V(1)
      T4 =T3-1.0
      FM(1)= P*PM(1,1,3)+T3*PM(1,2,3) + P*PM(1,3,3)-T3*PM(1,4,3)
      FM(2)= P*PM(1,1,3)+T4*PM(1,2,3) - P*PM(1,3,3)+T4*PM(1,4,3)
      FM(3)= P*PM(1,1,4)+T3*PM(1,2,4) + P*PM(1,3,4)-T3*PM(1,4,4)
      FM(4)= P*PM(1,1,4)+T4*PM(1,2,4) - P*PM(1,3,4)+T4*PM(1,4,4)
      DFAC=SC(1)/((FM(1)*FM(4)-FM(3)*FM(2))*P*P)
         A(LC,NS) = 0.0
         B(LC,NS) = 0.0
         C(LC,NS) = -FM(3)*DFAC
         D(LC,NS) =  FM(1)*DFAC
C     BACKSOLVE FOR THE OTHER A,B,C,D
      DO 91 K1=1,N
      A(LC,K1)=(PM(K1,1,3)*C(LC,NS)+PM(K1,1,4)*D(LC,NS))/SB(K1)
      B(LC,K1)=(PM(K1,2,3)*C(LC,NS)+PM(K1,2,4)*D(LC,NS))/SB(K1)
      C(LC,K1)=(PM(K1,3,3)*C(LC,NS)+PM(K1,3,4)*D(LC,NS))/SB(K1)
   91 D(LC,K1)=(PM(K1,4,3)*C(LC,NS)+PM(K1,4,4)*D(LC,NS))/SB(K1)
  100 CONTINUE
      RETURN
   99 ISW = -1
      DO 98 K1=1,N
      A(LC,K1)=0.0
```

```
      P(LC,K1)=0.0
      C(LC,K1)=0.0
   98 D(LC,K1)=0.0
      GO TO 100
      END
C
      SUBROUTINE BESSEL(NI,XI,Y)
C       ******SUBROUTINE BESSEL - N-LAYER ELASTIC SYSTEM ******
C
      DIMENSION PZ(6),QZ(6),P1(6),Q1(6),D(20)
      DATA PZ/1.0E0,-1.125E-4,2.8710938E-7,-2.8449658E-9,
     A3.9806841E-11,-1.1536133E-12/,  QZ/-5.0E-3,4.6875E-6,
     B-2.3255859E-8, 2.8307087E-10, -6.3912096E-12,  2.3124704E-12/,
     CP1/ 1.0E0,  1.875E-4,       -3.6914063E-7,  2.7713232E-9,
     D-4.5114421E-11,1.2750463E-12/, Q1/1.5E-2,  -6.5625E-6,
     E 2.8423828E-8,-3.2662024E-10,  7.1431166E-12, -2.5327056E-13/,
     F PI/3.1415927/
C
C
    9 N = NI
      X = XI
      IF (X-7.0) 10,10,160
C
   10 X2=X/2.0
      FAC=-X2*X2
      IF (N) 11,11,14
   11 C=1.0
      Y=C
      DO 13 I=1,34
      T=I
      C=FAC*C/(T*T)
      TEST=ABS (C) - 10.0**(-8)
      IF (TEST) 17,17,12
   12 Y=Y+C
   13 CONTINUE
   14 C=X2
      Y=C
      DO 16 I=1,34
      T=I
      C=FAC*C/(T*(T+1.0))
      TEST=ABS (C) - 10.0**(-8)
      IF (TEST) 17,17,15
   15 Y=Y+C
   16 CONTINUE
   17 RETURN
  160 IF (N) 161,161,164
C
C
  161 DO 162 I=1,6
      D(I) = PZ(I)
      D(I+10) = QZ(I)
  162 CONTINUE
```

```
      GO TO 163
C
  164 DO 165 I=1,6
      D(I) = P1(I)
      D(I+10) = Q1(I)
  165 CONTINUE
  163 CONTINUE
      T1 = 25.0/X
      T2=T1*T1
      P = D(6)*T2+D(5)
      DO 170 I=1,4
      J = 5-I
      P = P*T2+D(J)
  170 CONTINUE
      Q = D(16)*T2+D(15)
      DO 171 I=1,4
      J = 5-I
      Q = Q*T2+D(J+10)
  171 CONTINUE
      Q = Q*T1
C
      T4 =DSQRT (X*PI)
      T6 = SIN (X)
      T7 = COS (X)
C
      IF (N) 180,180,185
C
  180 T5 = ((P-Q)*T6 + (P+Q)*T7)/T4
      GO TO 99
  185 T5 = ((P+Q)*T6 - (P-Q)*T7)/T4
   99 Y = T5
      RETURN
      END
      SUBROUTINE PART
C       ******SUBROUTINE PART - N-LAYER ELASTIC SYSTEM ******
C
      COMMON   /RMCOY/RR(99),   ZZ(99),   E(15),    V(15),    HH(14),
     1                H(14),    AZ(396),  A(396,15),B(396,15),C(396,15),
     2                D(396,15),AJ(396),  RJ1(396), RJ0(396), TITLE(20),
     3                TEST(11), RZ(100),  X(15,4,4),SC(14),   FM(2,2),
     4               PM(14,4,4),R,        Z,        AR,       NS,
     5                N,        L,        ITN,      RSZ,      RSR,
     6                ROM,      RMU,      SF,       CSZ,      CST,
     7                CSR,      CTR,      COM,      CMU,      PSI,
     8                NLINE,    NOUTP,    NTEST,    I,        ITN4,
     9                K,        LC,       JT,       TZZ,      PR,
     A                PA,       P,        EP,       TIP,      TIM,
     B                T1,       T2,       T3,       T4,       T5,
     C                T6,       T2P,      T2M,      WA,       BJ1,
     D                BJ0,      ZF,       SZ1,      SZ2,      SG1,
     E                SG2,      PH,       PH2,      VK2,      VKP2,
     F                VK4,      VKP4,     VKK8,     RDT,      RDS
```

```
      DATA  G1/0.86113631/,G2/0.33998104/
    4 ZF = AR
      NTEST = 2
      IF (R) 8,8,9
    9 CONTINUE
      NTEST = AR/R + .0001
      IF (NTEST) 6,6,5
    6 CONTINUE
      NTEST = R/AR + .0001
      ZF = R
    5 CONTINUE
      NTEST = NTEST + 1
      IF (NTEST-10) 8,8,7
    7 CONTINUE
      NTEST = 10
    8 CONTINUE
C         ** COMPUTE POINTS FOR LEGENDRE-GAUSS INTEGRATION **
   15 K = 1
       ZF = 2.0*ZF
       SZ2 = 0.0
      DO 28 I=1,ITN
        SZ1 = SZ2
        SZ2 = BZ(I+1)/ZF
           SF = SZ2 - SZ1
           PP = SZ2 + SZ1
      SG1=SF*G1
      SG2=SF*G2
      AZ(K)=PP-SG1
      AZ(K+1)=PP-SG2
      AZ(K+2)=PP+SG2
      AZ(K+3)=PP+SG1
        K = K + 4
   28 CONTINUE
   40 RETURN
      END
C
      SUBROUTINE CALCIN
C     ******SUBROUTINE CALCIN - N-LAYER ELASTIC SYSTEM ******
C
      COMMON   /RMCOY/RR(99),   ZZ(99),   E(15),    V(15),    HH(14),
     1              H(14),    AZ(396),  A(396,15),B(396,15),C(396,15),
     2              D(396,15),AJ(396),  RJ1(396), RJ0(396), TITLE(28),
     3              TEST(11), BZ(100),  X(15,4,4),SG(14),   FM(2,2),
     4             PM(14,4,4),R,        Z,        AR,       NS,
     5              N,        L,        ITN,      RSZ,      RSR,
     6              ROM,      RMU,      SF,       CSZ,      CST,
     7              CSR,      CTR,      COM,      CMU,      PSI,
     8              NLINE,    NOUTP,    NTEST,    I,        ITN4,
     9              K,        LC,       JT,       TZZ,      PR,
     A              PA,       P,        EP,       TIP,      TIM,
     B              T1,       T2,       T3,       T4,       T5,
     C              T6,       T2P,      T2M,      WA,       BJ1,
```

```
     D                     BJ0,      ZF,       SZ1,      SZ2,      SG1,
     E                     SG2,      PH,       PH2,      VK2,      VKP2,
     F                     VK4,      VKP4,     VKK8,     RDT,      RDS
      DIMENSION W(4)
      DATA W/0.34785485,2*0.65214515,0.34785485/
C
    2 VL=2.0*V(L)
      FL=(1.0+V(L))/E(L)
      VL1=1.0-VL
      CSZ=0.0
      CST=0.0
      CSR=0.0
      CTR=0.0
      COM=0.0
      CMU=0.0
      NTS1 = NTEST + 1
      ITS = 1
      JT = 0
      ARP = AR
      IF (NOUTP) 4,4,5
    4 ARP = ARP*PSI
    5 CONTINUE
   10 DO 40 I=1,ITN
C     INITIALIZE THE SUB-INTEGRALS
      RSZ=0.0
      RST=0.0
      RSR=0.0
      RTR=0.0
      ROM=0.0
      RMU=0.0
C     COMPUTE THE SUB-INTEGRALS
           K = 4*(I-1)
      DO 30 J=1,4
           J1 = K + J
      P=AZ(J1)
      PZEXP=P*Z
      IF(PZEXP.GT.88.) PZEXP=88.
      IF(PZEXP.LT.-88.) PZEXP=-88.
      EP=EXP(PZEXP)
      T1=B(J1,L)*EP
      T2=D(J1,L)/EP
      T1P=T1+T2
      T1M=T1-T2
      T1=(A(J1,L)+B(J1,L)*Z)*EP
C
      T2=(C(J1,L)+D(J1,L)*Z)/EP
      T2P=P*(T1+T2)
      T2M=P*(T1-T2)
      WA=AJ(J1)*W(J)
      IF (R) 20,20,15
   15 BJ1=RJ1(J1)*P
      BJ0=RJ0(J1)*P
```

```
      RSZ=RSZ+WA*P*BJ0*(VL1*T1P-T2M)
      ROM=ROM+WA*EL*BJ0*(2.0*VL1*T1M-T2P)
      RTR=RTR+WA*P*BJ1*(VL*T1M+T2P)
      RMU=RMU+WA*EL*BJ1*(T1P+T2M)
      RSR=RSR+WA*(P*BJ0*((1.0+VL)*T1P+T2M)-BJ1*(T1P+T2M)/R)
      RST=RST+WA*(VL*P*BJ0*T1P+BJ1*(T1P+T2M)/R)
      GO TO 30
C     SPECIAL ROUTINE FOR R = ZERO
   20 PP=P*P
      RSZ=RSZ+WA*PP*(VL1*T1P-T2M)
      ROM=ROM+WA*EL*P*(2.0*VL1*T1M-T2P)
      RST=RST+WA*PP*((VL+0.5)*T1P+0.5*T2M)
      RSR=RST
   30 CONTINUE
C
       SF = (AZ(K+4) - AZ(K+1))/1.7222726
      CSZ=CSZ+RSZ*SF
      CST=CST+RST*SF
      CSR=CSR+RSR*SF
      CTR=CTR+RTR*SF
      COM=COM+ROM*SF
      CMU=CMU+RMU*SF
         RSZ = 2.0*RSZ*R*SF
      TESTH = ABS (RSZ)-10.0**(-4)
      IF (ITS-NTS1 ) 31,32,32
   31 CONTINUE
      TEST(ITS) = TESTH
      ITS = ITS+1
      GO TO 40
   32 CONTINUE
      TEST(NTS1) = TESTH
      DO 33 J = 1,NTEST
      IF (TESTH-TEST(J)) 35,36,35
   35 CONTINUE
      TESTH = TEST(J)
   36 CONTINUE
      TEST(J) = TEST(J+1)
   33 CONTINUE
      IF (TESTH) 50,50,40
   40 CONTINUE
      JT = 1
   50 CSZ=CSZ*ARP
      CST=CST*ARP
      CTR=CTR*ARP
      CSR=CSR*ARP
      OCTT1=(SQRT((CSZ-CST)**2+ (CSZ-CSR)**2+(CST-CSR)**2))/3.
      OCTT2=(CSZ+CST+CSR)/3.
      OCTT3=OCTT1/OCTT2
      DEVTOR=CSZ-(CST+CSR)/2.
      COM=COM*ARP
      CMU=CMU*ARP
      BSTS = CSZ+CST+CSR
```

```
      VSTR = (CSZ-V(L)*(CST+CSR))/E(L)
      BST  = BSTS * (1.0-2.0*V(L))/E(L)
      IF (TZZ) 72,72,71
   71 Z = -Z
   72 CONTINUE
      RDS=(CSR-V(L)*(CSZ+CST))/E(L)
      SST=2.0*(1.0+V(L))*CTR/E(L)
      RDT = (CST - V(L) * (CSZ + CSR))/E(L)
      WRITE (6,315) R,Z,CSZ,CST,CSR,CTR,BSTS,OCTT1,OCTT2,OCTT3,DEVTOR
      WRITE (6,316) VSTR,RDT,RDS,SST,BST
      WRITE (6,317) COM
  315 FORMAT (1H ,F5.1,F6.1,  7H STRESS    ,1P5E12.3,2X,0P4F10.2)
  316 FORMAT (13X, 6HSTRAIN    ,1P5E12.3)
  317 FORMAT (13X, 6HDISPLT    ,1PE12.3)
      NLINE = NLINE + 3
   99 RETURN
      END
C
C
      SUBROUTINE COES (KIN)
C      USED FOR 5 OR FEWER LAYERS
C
      COMMON   /RMGOV/RR(99),   ZZ(99),    E(15),     V(15),     HH(14),
     1                H(14),     AZ(396),   A(396,15),B(396,15),C(396,15),
     2                D(396,15), AJ(396),   RJ1(396), RJ0(396),  TITLE(20),
     3                TEST(11),  BZ(100),   X(15,4,4),SC(14),    FM(2,2),
     4                PM(14,4,4),R,         Z,        AR,        NS,
     5                N,         L,         ITN,      RSZ,       RSR,
     6                ROM,       RMU,       SF,       CSZ,       CST,
     7                CSR,       CTR,       COM,      CMU,       PSI,
     8                NLINE,     NOUTP,     NTEST,    I,         ITN4,
     9                K,         LC,        JT,       TZZ,       PR,
     A                PA,        P,         EP,       TIP,       TIM,
     B                T1,        T2,        T3,       T4,        T5,
     C                T6,        T2P,       T2M,      WA,        BJ1,
     D                BJ0,       ZF,        SZ1,      SZ2,       SG1,
     E                SG2,       PH,        PH2,      VKP,       VKP2,
     F                VK4,       VKP4,      VKK8,     RDT,       RDS
C
      REAL SV1(4,2),CV1(2,1),SV2(4,4),CV2(2,2),SV3(4,8),CV3(2,4),
     1     SV4(4,16),CV4(2,8),T(8)
      INTEGER  NT(14)
          LC = KIN                                                      COEE0380
CS-MX       SET UP MATRIX X =D|*M|*K|*K*M*D                            COEE0390
C     COMPUTE THE MATRICES X(K)                                         COEE0400
      DO 10 K=1,N                                                       COEE0410
      T1=E(K)*(1.0+V(K+1))/(E(K+1)*(1.0+V(K)))                          COEE0420
      T1M=T1-1.0                                                        COEE0430
      PH=P*H(K)                                                         COEE0440
      PH2=PH*2.0                                                        COEE0450
      VKP=2.0*V(K)                                                      COEE0460
      VKP2=2.0*V(K+1)                                                   COEE0470
```

```
      VK4=2.0*VK2
      VKP4=2.0*VKP2
      VKK8=8.0*V(K)*V(K+1)
C
      X(K,1,1)=VK4-3.0-T1
      X(K,2,1)=0.0
      X(K,3,1)=T1M*(PH2-VK4+1.0)
      X(K,4,1)=-2.0*T1M*P
C
      T3=PH2*(VK2-1.0)
      T4=VKK8+1.0-3.0*VKP2
      T5=PH2*(VKP2-1.0)
      T6=VKK8+1.0-3.0*VK2
C
      X(K,1,2)=(T3+T4-T1*(T5+T6))/P
      X(K,2,2)=T1*(VKP4-3.0)-1.0
      X(K,4,2)=T1M*(1.0-PH2-VKP4)
C
      X(K,3,4)=(T3-T4-T1*(T5-T6))/P
C
      T3=PH2*PH-VKK8+1.0
      T4=PH2*(VK2-VKP2)
C
      X(K,1,4)=(T3+T4+VKP2-T1*(T3+T4+VK2))/P
      X(K,3,2)=   (-T3+T4-VKP2+T1*(T3-T4+VK2))/P
C
      X(K,1,3)=T1M*(1.0-PH2-VK4)
      X(K,2,3)=2.0*T1M*P
      X(K,3,3)=VK4-3.0-T1
      X(K,4,3)=0.0
C
      X(K,2,4)=T1M*(PH2-VKP4+1.0)
      X(K,4,4)=T1*(VKP4-3.0)-1.0
   10 CONTINUE
C     COMPUTE THE PRODUCT MATRICES PM
      SC(N)=4.0*(V(N)-1.0)
      IF (N-2) 13,11,11
   11 DO 12 K1=2,N
      M=NS-K1
      SC(M)=SC(M+1)*4.0*(V(M)-1.0)
   12 CONTINUE
   13 CONTINUE
C
      K = N
      DO 15 M=1,4
      DO 14 J=1,2
   14 SV1(M,J) = X(K,M,J+2)
   15 CONTINUE
      CV1(1,1) = -2.0*P*H(K)
      CV1(2,1) = 0.0
      K = K-1
      IF(K) 50,50,20
```

```
C
   20 CONTINUE                                              COEE1020
      DO 22 J=1,2                                           COEE1030
      J1 = J+J                                              COEE1040
      T(1) = SV1(1,J)                                       COEE1050
      T(2) = SV1(2,J)                                       COEE1060
      T(3) = SV1(3,J)                                       COEE1070
      T(4) = SV1(4,J)                                       COEE1080
      DO 21 M=1,4                                           COEE1090
      SV2(M,J1-1) = X(K,M,1)*T(1)+X(K,M,2)*T(2)             COEE1100
   21 SV2(M,J1) =   X(K,M,3)*T(3)+X(K,M,4)*T(4)             COEE1110
   22 CONTINUE                                              COEE1120
      T(1) = CV1(1,1)                                       COEE1130
      T(2) = -2.0*P*H(K)                                    COEE1140
      CV2(1,1) = T(1)                                       COEE1150
      CV2(1,2) = T(2)                                       COEE1160
      CV2(2,1) = T(1)-T(2)                                  COEE1170
      CV2(2,2) = 0.0                                        COEE1180
      K = K-1                                               COEE1190
      IF (K) 50,50,30                                       COEE1200
C                                                           COEE1210
   30 CONTINUE                                              COEE1220
      DO 34 J=1,4                                           COEE1230
      J1 = J                                                COEE1240
      IF (J1-2) 32,32,31                                    COEE1250
   31 J1 = J1+2                                             COEE1260
   32 CONTINUE                                              COEE1270
      T(1) = SV2(1,J)                                       COEE1280
      T(2) = SV2(2,J)                                       COEE1290
      T(3) = SV2(3,J)                                       COEE1300
      T(4) = SV2(4,J)                                       COEE1310
      DO 33 M=1,4                                           COEE1320
      SV3(M,J1) = X(K,M,1)*T(1)+X(K,M,2)*T(2)               COEE1330
   33 SV3(M,J1+2) = X(K,M,3)*T(3)+X(K,M,4)*T(4)             COEE1340
   34 CONTINUE                                              COEE1350
      T(1) = -2.0*P*H(K)                                    COEE1360
      DO 35 J=1,2                                           COEE1370
      CV3(1,J) = CV2(1,J)                                   COEE1380
      CV3(2,J) = CV2(1,J)-T(1)                              COEE1390
      CV3(1,J+2) = CV2(2,J)*T(1)                            COEE1400
      CV3(2,J+2) = CV2(2,J)                                 COEE1410
   35 CONTINUE                                              COEE1420
      K = K-1                                               COEE1430
      IF (K) 50,50,40                                       COEE1440
C                                                           COEE1450
   40 CONTINUE                                              COEE1460
      DO 42 J=1,4                                           COEE1470
      T(1) = SV3(1,J)                                       COEE1480
      T(2) = SV3(2,J)                                       COEE1490
      T(3) = SV3(3,J)                                       COEE1500
      T(4) = SV3(4,J)                                       COEE1510
      T(5) = SV3(1,J+4)                                     COEE1520
                                                            COEE1530
```

```
      T(6) = SV3(2,J+4)                                          COEE1540
      T(7) = SV3(3,J+4)                                          COEE1550
      T(8) = SV3(4,J+4)                                          COEE1560
      DO 41 M=1,4                                                COEE1570
      SV4(M,J) = X(K,M,1)*T(1)+X(K,M,2)*T(2)                     COEE1580
      SV4(M,J+4) = X(K,M,3)*T(3)+X(K,M,4)*T(4)                   COEE1590
      SV4(M,J+8) = X(K,M,1)*T(5)+X(K,M,2)*T(6)                   COEE1600
   41 SV4(M,J+12) = X(K,M,3)*T(7)+X(K,M,4)*T(8)                  COEE1610
   42 CONTINUE                                                   COEE1620
      T(1) = -2.0*P*H(K)                                         COEE1630
      DO 43 J=1,4                                                COEE1640
      CV4(1,J) = CV3(1,J)                                        COEE1650
      CV4(2,J) = CV3(1,J)-T(1)                                   COEE1660
      CV4(1,J+4) = CV3(2,J)+T(1)                                 COEE1670
      CV4(2,J+4) = CV3(2,J)                                      COEE1680
   43 CONTINUE                                                   COEE1690
C                                                                COEE1700
   50 CONTINUE                                                   COEE1710
      NT(1) = 1                                                  COEE1720
      DO 51 K=2,N                                                COEE1730
   51 NT(K) = NT(K-1)+NT(K-1)                                    COEE1740
      DO 80 K=1,N                                                COEE1750
      K1 = NS-K                                                  COEE1760
      DO 52 M=1,4                                                COEE1770
      PM(K1,M,1) = 0.0                                           COEE1780
      PM(K1,M,2) = 0.0                                           COEE1790
   52 CONTINUE                                                   COEE1800
      I1 = NT(K)                                                 COEE1810
      DO 80 M=1,I1                                               COEE1820
      I2 = M+I1                                                  COEE1830
      GO TO (61,62,63,64),K                                      COEE1840
   61 CONTINUE                                                   COEE1850
      T(3) = CV1(1,M)                                            COEE1860
      T(4) = CV1(2,M)                                            COEE1870
      GO TO 65                                                   COEE1880
   62 CONTINUE                                                   COEE1890
      T(3) = CV2(1,M)                                            COEE1900
      T(4) = CV2(2,M)                                            COEE1910
      GO TO 65                                                   COEE1920
   63 CONTINUE                                                   COEE1930
      T(3) = CV3(1,M)                                            COEE1940
      T(4) = CV3(2,M)                                            COEE1950
      GO TO 65                                                   COEE1960
   64 CONTINUE                                                   COEE1970
      T(3) = CV4(1,M)                                            COEE1980
      T(4) = CV4(2,M)                                            COEE1990
   65 CONTINUE                                                   COEE2000
      T(1) = 0.0                                                 COEE2010
      T(2) = 0.0                                                 COEE2020
      IF (T(3)+68.0) 67,66,66                                    COEE2030
   66 T(1) = EXP(T(3))                                           COEE2040
   67 IF (T(4)+68.0) 69,68,68                                    COEE2050
```

```
   68 T(2) = EXP(T(4))                                     COEE2060
   69 CONTINUE                                             COEE2070
      DO 80 J=1,2                                          COEE2080
      GO TO (71,72,73,74),K                                COEE2090
   71 CONTINUE                                             COEE2100
      T(3) = SV1(J,M)                                      COEE2110
      T(4) = SV1(J,I2)                                     COEE2120
      T(5) = SV1(J+2,M)                                    COEE2130
      T(6) = SV1(J+2,I2)                                   COEE2140
      GO TO 75                                             COEE2150
   72 T(3) = SV2(J,M)                                      COEE2160
      T(4) = SV2(J,I2)                                     COEE2170
      T(5) = SV2(J+2,M)                                    COEE2180
      T(6) = SV2(J+2,I2)                                   COEE2190
      GO TO 75                                             COEE2200
   73 T(3) = SV3(J,M)                                      COEE2210
      T(4) = SV3(J,I2)                                     COEE2220
      T(5) = SV3(J+2,M)                                    COEE2230
      T(6) = SV3(J+2,I2)                                   COEE2240
      GO TO 75                                             COEE2250
   74 T(3) = SV4(J,M)                                      COEE2260
      T(4) = SV4(J,I2)                                     COEE2270
      T(5) = SV4(J+2,M)                                    COEE2280
      T(6) = SV4(J+2,I2)                                   COEE2290
   75 CONTINUE                                             COEE2300
C                                                          COEE2310
      PM(K1,J,1) = PM(K1,J,1)+T(1)*T(3)                    COEE2320
      PM(K1,J,2) = PM(K1,J,2)+T(1)*T(4)                    COEE2330
      PM(K1,J+2,1) = PM(K1,J+2,1)+T(2)*T(5)                COEE2340
      PM(K1,J+2,2) = PM(K1,J+2,2)+T(2)*T(6)                COEE2350
   80 CONTINUE                                             COEE2360
C     SOLVE FOR C(NS) AND D(NS)                            COEE2380
      V2=2.0*V(1)                                          COEE2390
      V21=V2-1.0                                           COEE2400
      DO 90 J=1,2                                          COEE2410
      FM(1,J)=P*PM(1,1,J)+V2*PM(1,2,J)+P*PM(1,3,J)-V2*PM(1,4,J)  COEE2420
   90 FM(2,J)=P*PM(1,1,J)+V21*PM(1,2,J)-P*PM(1,3,J)+V21*PM(1,4,J) COEE2430
      DFAC=SC(1)/((FM(1,1)*FM(2,2)-FM(2,1)*FM(1,2))*P*P)   COEE2440
         A(LC,NS) = 0.0                                    COEE2450
         B(LC,NS) = 0.0                                    COEE2460
         C(LC,NS) = -FM(1,2)*DFAC                          COEE2470
         D(LC,NS) =  FM(1,1)*DFAC                          COEE2480
C     BACKSOLVE FOR THE OTHER A,B,C,D                      COEE2500
      DO 91 K1=1,N                                         COEE2510
      A(LC,K1)=(PM(K1,1,1)*C(LC,NS)+PM(K1,1,2)*D(LC,NS))/SC(K1)  COEE2520
      B(LC,K1)=(PM(K1,2,1)*C(LC,NS)+PM(K1,2,2)*D(LC,NS))/SC(K1)  COEE2530
      C(LC,K1)=(PM(K1,3,1)*C(LC,NS)+PM(K1,3,2)*D(LC,NS))/SC(K1)  COEE2540
   91 D(LC,K1)=(PM(K1,4,1)*C(LC,NS)+PM(K1,4,2)*D(LC,NS))/SC(K1)  COEE2550
      RETURN                                               COEE2570
      END                                                  COEE2580
C
      SUBROUTINE WHEELS (RAD,NDEP,DEPTH,S,LAYER,E,V,RADII,NLAY,PSI,ZP,
```

```
      1NPAGE)
      DIMENSION  RAD(9),DEPTH(15),LAYER(15),S(5,9,15),XW(16),YW(16),
     1           ZP(15),SS(5),E(6),V(6),R(16),RINP(16),COSR(16),
     2           J1(16),J2(16),J3(16),H(16),R1(16)
7     CONTINUE
9005  FORMAT(//,5X,34HDATA FROM CHEVRON N-LAYER PROGRAM ,/,68H  RAL.US
     1   DEPTH   VERTICAL  TANGENT   RADIAL    SHEAR   DISPLACE/)
      SS(1)=0.0
9006  FORMAT(              6(F9.3,1X),E10.3)
10    CONTINUE
C TABLE 11
      READ(5,9020) XNW
      NW=XNW
9020  FORMAT(V)
8055  IF(NW.EQ.0)GO TO 9999                                      AUST1/74
C TABLE 12
      READ(5,9020) (XW(I),YW(I),I=1,NW)
9021  FORMAT (2F10.0)                                                 253
8060  CONTINUE                                                   AUST1/74
      NLINE=NLAY+17
      WRITE(6,9003)
9003  FORMAT(//,5X,45HTIRE   X-ORD     Y-ORD     RADIUS   CON.PRESS/)
      DO 9 I=1,NW
      WRITE(6,9004) I,XW(I),YW(I),RADII,PSI
9004  FORMAT(5X,I3,4(3X,F7.2))
9     CONTINUE
      NLINE=NLINE+(NW+3)
    7 CONTINUE
      ND=NDEP
9030  FORMAT(F10.0)                                              AUST1/74
9036  FORMAT(8F10.0)                                             AUST1/74
7000  CONTINUE                                                   AUST1/74
8070  CONTINUE                                                   AUST1/74
12    CONTINUE
C TABLE 13
      READ(5,9020) XNGRID
      NGRID=XNGRID
      DO 6000 KK=1,NGRID
      IF(KK-1)6001,6001,6002
6002  NPAGE=NPAGE+1
      WRITE(6,4900) NPAGE
      WRITE(6,6005)KK
6005  FORMAT(//'........  GRID NO. ',I3//)
      NLINE=12
6001  CONTINUE
C TABLE 14
      READ(5,9020) XXP, YP, DX, DY, XNUMX, XNUMY
      NUMX=XNUMX
      NUMY=XNUMY
9037  FORMAT(6F10.0)                                             AUST1/74
      DO 199 II=1,NUMY
      XP=XXP
```

```
      DO 299 JJ=1,NUMX
 9022 FORMAT (2F10.0,I5,5X,10F5.0)
      IF(ND.EQ.0)GO TO 1
      IF(NLINE-54)4001,4002,4002
 4002 NPAGE=NPAGE+1
      WRITE(6,4900)NPAGE
 4900 FORMAT(1H1//1H0,25(1H*),33X,5HPAGE ,I3,33X,25(1H*))
      NLINE=8
 4001 CONTINUE
      WRITE(6,9010) XP,YP
      NLINE=NLINE+4
 9010 FORMAT(//,5X,12HX-ORDINATE =,F7.2,5X,12HY-ORDINATE =,F7.2,/,1X,
     1 5HDEPTH, 14X,2HZZ,10X,2HXX,10X,2HYY,10X,2HXY,10X,2HXZ,10X,2HYZ,
     26X,8HOCTSHEAR,3X,9HOCTSTRESS,7X,5HRATIO,2X,8HDEVIATOR)
      DO 50 I=1,NW
      IF((XP-XW(I)).EQ.0.0.AND.(YP-YW(I)).EQ.0.0) GO TO 15
      R(I)=SQRT((XW(I)-XP)**2 + (YW(I)-YP)**2)
      SINP(I)=(YP-YW(I))/R(I)
      CORP(I)=(XP-XW(I))/R(I)
      GO TO 18
   15 CONTINUE
      CORP(I)=0.0
      SINP(I)=0.0
      R(I)=0.0
   18 CONTINUE
      IF(R(I).GE.RAD(1).AND.R(I).LE.RAD(3))GOTO 20
      IF(R(I).GT.RAD(3).AND.R(I).LE.RAD(5))GO TO 25
      IF(R(I).GT.RAD(5).AND.R(I).LE.RAD(7)) GO TO 30
      IF(R(I).GT.RAD(7).AND.R(I).LE.RAD(9)) GO TO 35
      J1(I) =  9
      J2(I)=9
      J3(I)=9
      H(I)   = RAD(9)
      R1(I)  = RAD(9)
      GO TO 45
   20 J1(I)=1
      J2(I)=2
      J3(I)=3
      H(I)   = RAD(2) - RAD(1)
      R1(I)  = RAD(1)
      GO TO 45
   25 J1(I)=3
      J2(I)=4
      J3(I)=5
      H(I)   = RAD(4) - RAD(3)
      R1(I)  = RAD(3)
      GO TO 45
   30 J1(I)=5
      J2(I)=6
      J3(I)=7
      H(I)   = RAD(6) - RAD(5)
      R1(I)  = RAD(5)
```

```
      GO TO 45
35    J1(I)=7
      J2(I)=8
      J3(I)=9
      H(I) = RAD(8) - RAD(7)
      R1(I) = RAD(7)
45    CONTINUE
50    CONTINUE
      DO 100 I=1,ND
      DO 60 J=1,NDEP
      LD=J
      IF(ZP(I).EQ.DEPTH(J)) GO TO 65
60    CONTINUE
      WRITE(6,9100) ZP(I)
9100  FORMAT(/,14H DEPTH OMITTED,F10.1)
      GO TO 100
65    CONTINUE
      XX=0.0
      YY=0.0
      XY=0.0
      ZZ=0.0
      XZ=0.0
      YZ=0.0
      DISP=0.0
      DO 99 J=1,NW
      K1=J1(J)
      K2=J2(J)
      K3=J3(J)
      HH=H(J)
      X1=R1(J)
      X=R(J)
      DO 98 K=1,5
      Y1=S(K,K1,LD)
      Y2=S(K,K2,LD)
      Y3=S(K,K3,LD)
      IF(K1.NE.9) GO TO 97
      Y2 = Y1/10.0
      Y3 = Y1/100.0
97    CONTINUE
      CALL FIT (Y1,Y2,Y3,X1,X,HH,SS,K)
98    CONTINUE
      IF(ZP(I).EQ.0.0)  GO TO 95
      ZZ=SS(1) + ZZ
      GO TO 96
95    IF(R(J).LE.RADII) ZZ=-PSI+ZZ
96    CONTINUE
      IF(SINP(J).EQ.0.0.AND.COSP(J).EQ.0.0)XX =SS(2) + XX
      IF(SINP(J).EQ.0.0.AND.COSP(J).EQ.0.0)YY =SS(2) + YY
      XX=SS(3)*COSP(J)*COSP(J)+SS(2)*SINP(J)*SINP(J)+XX
      YY=SS(3)*SINP(J)*SINP(J)+SS(2)*COSP(J)*COSP(J)+YY
      XY=SS(3)*SINP(J)*COSP(J)+SS(2)*SINP(J)*COSP(J)+XY
      XZ=SS(4)*COSP(J)+XZ
```

```
      YZ=SS(4)*SINP(J)+YZ
      DISP=SS(5)+DISP
      OCTT=(SQRT((ZZ-XX)**2+(ZZ-YY)**2+(XX-YY)**2))/3.
       OCTS=(ZZ+XX+YY)/3.
      OCTR=OCTT/OCTS
      DEVTOR=ZZ-(XX+YY)/2.
99    CONTINUE
      WRITE(6,9011) ZP(I),ZZ,XX,YY,XY,XZ,YZ,OCTT,OCTS,OCTR,DEVTOR
      L=LAYER(LD)
      ZS=(ZZ-V(L)*(XX+YY))/E(L)
      XS=(XX-V(L)*(ZZ+YY))/E(L)
      YS=(YY-V(L)*(ZZ+XX))/E(L)
      G=E(L)/(2.0*(1.0+V(L)))
      XYS=XY/G
      XZS=XZ/G
      YZS=YZ/G
      WRITE(6,9013) ZS,XS,YS,XYS,XZS,YZS
      WRITE(6,9012) DISP
      NLINE=NLINE+3
      IF(NLINE-54)4008,4009,4009
4009  NPAGE=NPAGE+1
      WRITE(6,4900)NPAGE
      NLINE=8
4008  CONTINUE
9011  FORMAT(F6.1,6HSTRESS,2X, 8(1X,F9.2,2X),F6.2,4X,F9.2)
9012  FORMAT(6X,6HDISPLT,2X,1PE11.3)
9013  FORMAT(6X,6HSTRAIN,2X,6(1X,1PE10.3,1X))
9014  FORMAT(20X,6HSTRESS,6X,6HSTRAIN,6X,6HX-COMP,6X,6HY-COMP,
     1        6X,6HZ-COMP)
9027  FORMAT(57H VALUES AND DIRECTIONS OF PRINCIPAL STRESSES AND STRAINS
     1   /)
9028  FORMAT(1H1)
9015  FORMAT(/,27H *** POINT COORDINATES  X =,F7.2,4X,3HY =,F7.2,
     +4X,3HZ =,F7.2,4X,4H ***,/)
9110  FORMAT(3F4.1,I3,6E9.2)
9111  FORMAT(20X,E10.3)
100   CONTINUE
      XP=XP+DX
200   CONTINUE
      YP=YP+DY
100   CONTINUE
6000  CONTINUE
9999  CONTINUE
      WRITE(6,9038)
9038  FORMAT(/,10X,16H THIS IS THE END)
      RETURN
      END
C
      SUBROUTINE FIT (Y1,Y2,Y3,X1,X,HH,SS,K)
      DIMENSION SS(5)
      D1=Y2-Y1
      D2=Y3-2.*Y2+Y1
```

```
      U=(X-X1)/HW
      SS(K)=Y1+D1*U+D2*U*(U-1.)/2.
      RETURN
      END
C
      SUBROUTINE MOD(S,E,LY,KAR,KARTER,IZ,IR,ZZ,TOL,NLIN,ITRIP,IZZ)
      DIMENSION S(5,9,15),E(15),LY(15),KAR(15),KARTER(15,5),ZZ(15),
     1DIF(2,15),ENEW(15),FACTOR(15),IZZ(15)
      REAL KARTER
      IF(ITRIP.EQ.1) PER=TOL/100.0
      WRITE(6,4002)ITRIP
 4002 FORMAT(//,10X,28HITERATION DATA FOR CYCLE NO.,I5,/
     117X,5HLAYER,7X,7HINIT. E,8X,6HCOMP E,9X,7HNEW   E ,
     28X,6HSTRESS,10X,8HRELAX. F,/)
      DO 10 I=1,5
      KK=0
      DO 10 J=1,IZ
      SUM=0.
      DO 5 L=1,IR
      SUME=0.
      IZZZ=IZZ(J)
       IF(I .GT.1)KK=KK-IZZZ
      DO 6 K=1,IZZZ
      KK=KK+1
      IF(I.GT.3)GO TO 7
      S(I,L,KK)=S(I,L,KK)-ZZ(KK)*.078
      GO TO 8
    7 CONTINUE
      IF(I.EQ.4)GO TO 8
      S(I,L,KK)=S(I,L,KK)-3.*ZZ(KK)*.078
    8 CONTINUE
    6 SUME=S(I,L,KK)+SUME
      SUME=SUME/IZZZ
    5 SUM=SUM+SUME
   10 S(I,1,J)=-SUM/IR
  100 CONTINUE
      ISTOP=1
      DO 110 I=1,IZ
      L=LY(I)
      GO TO (200,300,400,500),KAR(I)
  200 CONTINUE
C           E IS FUNCTION OF S1+S2+S3
C           COMPUTE OVERBURDEN
      SUM=S(1,1,I)+S(2,1,I)+S(3,1,I)
      IF(SUM.LT.0)SUM=1.0
      ENEW(L)=KARTER(I,1)*SUM**KARTER(I,2)
      GO TO 600
  300 CONTINUE
  400 CONTINUE
  500 CONTINUE
C      FOR SUBGRADE SOILS E IS A FUNCTION OF THE DEVIATOR STRESS
      SUM=S(2,1,I)
```

```
      IF(S(2,1,I).GT.S(3,1,I))SUM=S(3,1,I)
      SUM=S(1,1,I)-SUM
      IF(SUM.GT.KARTER(I,1)) GO TO 310
      ENEW(L)=KARTER(I,2)+KARTER(I,3)*(KARTER(I,1)-SUM)
      GO TO 600
  310 CONTINUE
      ENEW(L)=KARTER(I,2)+KARTER(I,4)*(SUM-KARTER(I,1))
  600 CONTINUE
      IF(ITRIP.GT.1) GO TO 115
      DIF(1,I)=E(L)-ENEW(L)
      EX=(E(L)+ENEW(L))/2.
      FACTOR(I)=1.0
      GO TO 150
  115 CONTINUE
      IF(ITRIP.GT.2)GO TO 116
      DIF(2,I)=E(L)-ENEW(L)
      GO TO 120
  116 CONTINUE
      DIF(1,I)=DIF(2,I)
      DIF(2,I)=E(L)-ENEW(L)
  120 CONTINUE
      DIFF=DIF(1,I)*DIF(2,I)
      IF(DIFF.LT.0.0)FACTOR(I)=FACTOR(I)/2.0
      IF(DIFF.GT.0.0)FACTOR(I)=FACTOR(I)*2.0
      EX=(E(L)+FACTOR(I)*ENEW(L))/(1.0+FACTOR(I))
  150 CONTINUE
 4000 FORMAT(10X,15H ITERATION DATA,//)
      WRITE(6,4001)LY(I),E(L),ENEW(L),EX,SUM,FACTOR(I)
 4001 FORMAT(10X,I10,3F15.0,2F15.3)
      TOL=E(L)*PER
      E(L)=EX
      IF(ITRIP.EQ.1) GO TO 170
      IF(ABS(DIF(1,I)).LT.TOL.AND.ABS(DIF(2,I)).LT.TOL) GO TO 160
  170 CONTINUE
      ISTOP=-1
  160 CONTINUE
  110 CONTINUE
      IF(ITRIP.GT.NLIN.AND.ISTOP.LT.0)GO TO 9999
      IF(ISTOP.GT.0) NLIN=0
      RETURN
 9999 CONTINUE
      WRITE(6,9998)
 9998 FORMAT (38H MAXIMUM NUMBER OF ITERATIONS EXCEEDED )
      CALL EXIT
      RETURN
      END
```

EXAMPLE COMPUTATION

INPUT

5. To illustrate the use of the program, response parameters will be computed for the idealized pavement section shown in Figure J1. The

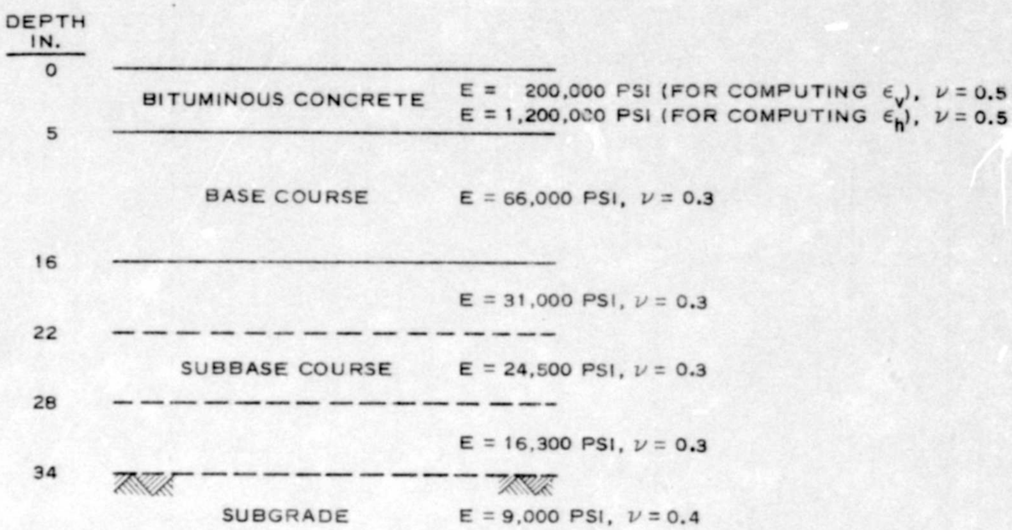


Figure J1. Idealized pavement section

gross aircraft loading is 200,000 lb on a dual-tandem assembly having a 21- by 46-in. spacing and a 160-psi tire contact pressure. The gear coordinate system shown in Figure J2 is the reference coordinate system. For the

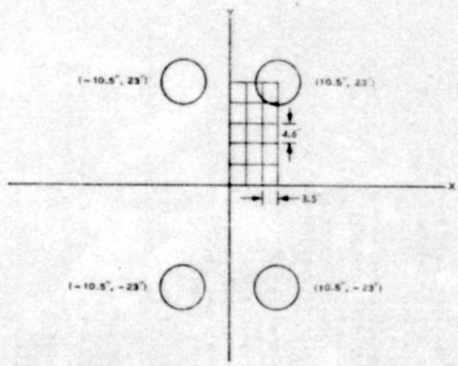


Figure J2. Gear coordinates and guide for computational points

computational points, a grid is used which encompasses one quadrant from the origin to the tire. The computational points are the intersections of the grid lines, as shown in Figure J2. The computational depths are 5 and 34 in. (each of which is an interface), resulting in a total of four computational depths. The input problem data coded in a form for keypunching are presented in Table J15.

OUTPUT

6. The output format is such that the first part of the output is the data for establishing the relationship for pavement response as a function of offset distance. The second part of the output shows the results of rotation and superposition of the stresses as computed for multiple-wheel gears. The output for the example problem is shown in the listing that follows Table J15.

Table J15

GENERAL PURPOSE DATA FORM

PROGRAM

REQUESTED BY | PREPARED BY | CHECKED BY | DATE | PAGE OF

1–10	11–20	21–30	31–40	41–50	51–60	61–70	71–80
START							
	EXAMPLE DE	SIGN PROBL	EM				
23750.	160.						
6.							
200000.	0.5	66000.	0.3	31000.	0.3	24500.	0.3
16300	0.3	9000.	0.4				
5.	11.	6.	6.	6.			
0.							
2.							
5.	34.						
4.							
-10.5	-23.						
10.5	-23.						
-10.5	23.						
10.5	23.						
1.							
0.	0.	3.5	4.6	4.	6.		
END							

WES FORM NO. 1233
SEPTEMBER 1962

EXAMPLE DESIGN PROBLEM

THE PROBLEM PARAMETERS ARE

TOTAL LOAD.. 23750.00 LBS

TIRE PRESSURE.. 160.00 PSI

LOAD RADIUS.. 6.87 IN.

LAYER 1 HAS MODULUS 200000. POISSONS RATIO 0.500 AND THICKNESS 5.00 IN.
LAYER 2 HAS MODULUS 66000. POISSONS RATIO 0.300 AND THICKNESS 11.00 IN.
LAYER 3 HAS MODULUS 31000. POISSONS RATIO 0.300 AND THICKNESS 6.00 IN.
LAYER 4 HAS MODULUS 24500. POISSONS RATIO 0.300 AND THICKNESS 6.00 IN.
LAYER 5 HAS MODULUS 16300. POISSONS RATIO 0.300 AND THICKNESS 6.00 IN.
LAYER 6 HAS MODULUS 9000. POISSONS RATIO 0.400 AND IS SEMI-INFINITE.

R	Z		VERTICAL	TANGENT	RADIAL	SHEAR	BULK	OCTSHEAR	OCTSTRESS	RATIO
0.	-5.0	STRESS	-9.807E 01	6.941E 01	6.941E 01	0.	4.075E 01	78.95	13.58	5.81
		STRAIN	-8.374E-04	4.187E-04	4.187E-04	0.	0.			
		DISPLT	4.068E-02							
0.	5.0	STRESS	-9.807E 01	-2.552E 00	-2.552E 00	0.	-1.032E 02	45.03	-34.39	-1.31
		STRAIN	-1.463E-03	4.187E-04	4.187E-04	0.	-6.253E-04			
		DISPLT	4.068E-02							
0.	-34.0	STRESS	-3.963E 00	2.897E 00	2.897E 00	0.	1.831E 00	3.23	0.61	5.30
		STRAIN	-3.498E-04	1.973E-04	1.973E-04	0.	4.492E-05			
		DISPLT	2.362E-02							
0.	34.0	STRESS	-3.963E 00	3.181E-01	3.181E-01	0.	-3.327E 00	2.02	-1.11	-1.82
		STRAIN	-4.686E-04	1.973E-04	1.973E-04	0.	-7.393E-05			
		DISPLT	2.362E-02							
3.4	-5.0	STRESS	-8.671E 01	6.259E 01	5.496E 01	-2.216E 01	3.084E 01	68.65	10.28	6.68
		STRAIN	-7.274E-04	3.923E-04	3.351E-04	-3.324E-04	0.			
		DISPLT	3.929E-02							
3.4	5.0	STRESS	-8.671E 01	-1.414E 00	-4.322E 00	-2.216E 01	-9.244E 01	39.54	-30.81	-1.28
		STRAIN	-1.288E-03	3.923E-04	3.351E-04	-8.729E-04	-5.603E-04			
		DISPLT	3.929E-02							
3.4	-34.0	STRESS	-3.903E 00	2.856E 00	2.807E 00	-3.174E-01	1.760E 00	3.18	0.59	5.41
		STRAIN	-3.437E-04	1.954E-04	1.915E-04	-5.062E-05	4.319E-05			
		DISPLT	2.351E-02							
3.4	34.0	STRESS	-3.903E 00	3.120E-01	2.870E-01	-3.174E-01	-3.304E 00	1.98	-1.10	-1.80
		STRAIN	-4.603E-04	1.954E-04	1.915E-04	-9.873E-05	-7.343E-05			
		DISPLT	2.351E-02							
6.9	-5.0	STRESS	-5.052E 01	3.159E 01	-4.140E 00	-3.464E 01	-2.307E 01	33.62	-7.69	-4.37
		STRAIN	-3.212E-04	2.946E-04	2.663E-05	-5.196E-04	0.			
		DISPLT	3.558E-02							
6.9	5.0	STRESS	-5.052E 01	2.942E-01	-1.331E 01	-3.464E 01	-6.354E 01	21.48	-21.18	-1.01
		STRAIN	-7.063E-04	2.946E-04	2.663E-05	-1.364E-03	-3.851E-04			
		DISPLT	3.558E-02							

EXAMPLE DESIGN PROBLEM

R	Z		VERTICAL	TANGENT	RADIAL	SHEAR	BULK	OCTSHEAR	OCTSTRESS	RATIO
6.9	-34.0	STRESS	-3.733E 00	2.739E 00	2.555E 00	-6.037E-01	1.561E 00	3.01	0.52	5.78
		STRAIN	-3.265E-04	1.897E-04	1.750E-04	-9.630E-05	3.830E-05			
		DISPLT	2.319E-02							
6.9	34.0	STRESS	-3.733E 00	2.942E-01	1.997E-01	-6.037E-01	-3.239E 00	1.88	-1.08	-1.74
		STRAIN	-4.367E-04	1.897E-04	1.750E-04	-1.878E-04	-7.198E-05			
		DISPLT	2.319E-02							
10.3	-5.0	STRESS	-1.820E 01	7.919E-01	-4.462E 01	-2.515E 01	-6.203E 01	18.62	-20.68	-0.90
		STRAIN	1.859E-05	1.610E-04	-1.796E-04	-3.773E-04	0.			
		DISPLT	3.134E-02							
10.3	5.0	STRESS	-1.820E 01	-2.917E-02	-1.732E 01	-2.515E 01	-3.555E 01	8.37	-11.85	-0.71
		STRAIN	-1.969E-04	1.610E-04	-1.796E-04	-9.909E-04	-2.154E-04			
		DISPLT	3.134E-02							
10.3	-34.0	STRESS	-3.527E 00	2.651E 00	2.272E 00	-8.372E-01	1.396E 00	2.83	0.47	6.08
		STRAIN	-3.070E-04	1.857E-04	1.555E-04	-1.335E-04	3.425E-05			
		DISPLT	2.247E-02							
10.3	34.0	STRESS	-3.527E 00	3.049E-01	1.107E-01	-8.372E-01	-3.112E 00	1.76	-1.04	-1.70
		STRAIN	-4.104E-04	1.857E-04	1.555E-04	-2.605E-04	-6.915E-05			
		DISPLT	2.247E-02							
13.7	-5.0	STRESS	-6.266E 00	-7.835E 00	-3.979E 01	-1.534E 01	-5.389E 01	15.44	-17.96	-0.86
		STRAIN	8.772E-05	7.595E-05	-1.637E-04	-2.302E-04	0.			
		DISPLT	2.822E-02							
13.7	5.0	STRESS	-6.266E 00	-7.380E-01	-1.290E 01	-1.534E 01	-1.991E 01	4.97	-6.64	-0.75
		STRAIN	-3.293E-05	7.595E-05	-1.637E-04	-6.045E-04	-1.207E-04			
		DISPLT	2.822E-02							
13.7	-34.0	STRESS	-3.231E 00	2.430E 00	1.831E 00	-1.001E 00	1.030E 00	2.54	0.34	7.40
		STRAIN	-2.766E-04	1.748E-04	1.271E-04	-1.596E-04	2.527E-05			
		DISPLT	2.188E-02							
13.7	34.0	STRESS	-3.231E 00	2.639E-01	-4.309E-02	-1.001E 00	-3.010E 00	1.58	-1.00	-1.57
		STRAIN	-3.688E-04	1.748E-04	1.271E-04	-3.113E-04	-6.688E-05			
		DISPLT	2.188E-02							
20.6	-5.0	STRESS	-8.622E-01	-8.575E 00	-2.073E 01	-6.992E 00	-3.018E 01	8.17	-10.06	-0.81
		STRAIN	6.884E-05	1.114E-05	-7.999E-05	-1.049E-04	0.			
		DISPLT	2.386E-02							
20.6	5.0	STRESS	-8.822E-01	-1.310E 00	-5.937E 00	-6.992E 00	-8.129E 00	2.29	-2.71	-0.84
		STRAIN	1.957E-05	1.114E-05	-7.999E-05	-2.754E-04	-4.927E-05			
		DISPLT	2.386E-02							
20.6	-34.0	STRESS	-2.562E 00	1.941E 00	9.445E-01	-1.140E 00	3.229E-01	1.93	0.11	17.94
		STRAIN	-2.103E-04	1.488E-04	6.938E-05	-1.819E-04	7.924E-06			
		DISPLT	2.035E-02							
20.6	34.0	STRESS	-2.562E 00	1.838E-01	-3.269E-01	-1.140E 00	-2.705E 00	1.19	-0.90	-1.32
		STRAIN	-2.783E-04	1.488E-04	6.938E-05	-3.547E-04	-6.012E-05			
		DISPLT	2.035E-02							

R	Z		VERTICAL	TANGENT	RADIAL	SHEAR	BULK	OCTSHEAR	OCTSTRESS	RATIO
27.5	-5.0	STRESS	-2.882E-01	-6.918E 00	-1.127E 01	-3.991E 00	-1.847E 01	4.51	-6.16	-0.73
		STRAIN	4.402E-05	-5.700E-06	-3.833E-05	-5.986E-05	0.			
		DISPLT	2.078E-02							
27.5	5.0	STRESS	-2.882E-01	-1.371E 00	-3.027E 00	-3.991E 00	-4.686E 00	1.13	-1.56	-0.72
		STRAIN	1.562E-05	-5.700E-06	-3.833E-05	-1.572E-04	-2.840E-05			
		DISPLT	2.078E-02							
27.5	-34.0	STRESS	-1.965E 00	1.493E 00	2.564E-01	-1.103E 00	-2.158E-01	1.43	-0.07	-19.88
		STRAIN	-1.527E-04	1.230E-04	2.443E-05	-1.759E-04	-5.296E-06			
		DISPLT	1.865E-02							
27.5	34.0	STRESS	-1.965E 00	1.128E-01	-5.210E-01	-1.103E 00	-2.373E 00	0.87	-0.79	-1.10
		STRAIN	-2.002E-04	1.230E-04	2.443E-05	-3.431E-04	-5.273E-05			
		DISPLT	1.865E-02							
55.0	-5.0	STRESS	-7.150E-02	-2.823E 00	-1.796E 00	-8.110E-01	-4.690E 00	1.14	-1.56	-0.73
		STRAIN	1.119E-05	-9.446E-06	-1.744E-06	-1.217E-05	0.			
		DISPLT	1.324E-02							
55.0	5.0	STRESS	-7.150E-02	-7.537E-01	-3.626E-01	-8.110E-01	-1.188E 00	0.28	-0.40	-0.71
		STRAIN	3.991E-06	-9.446E-06	-1.744E-06	-3.195E-05	-7.199E-06			
		DISPLT	1.324E-02							
55.0	-34.0	STRESS	-6.723E-01	4.789E-01	-6.267E-01	-5.947E-01	-8.200E-01	0.53	-0.27	-1.95
		STRAIN	-3.852E-05	5.329E-05	-3.489E-05	-9.485E-05	-2.012E-05			
		DISPLT	1.285E-02							
55.0	34.0	STRESS	-6.723E-01	-2.675E-02	-5.936E-01	-5.947E-01	-1.293E 00	0.29	-0.43	-0.67
		STRAIN	-4.713E-05	5.329E-05	-3.489E-05	-1.850E-04	-2.872E-05			
		DISPLT	1.285E-02							
82.5	-5.0	STRESS	-8.686E-03	-1.251E 00	-1.364E-01	-1.881E-01	-1.396E 00	0.56	-0.47	-1.20
		STRAIN	3.424E-06	-5.691E-06	2.466E-06	-2.822E-06	0.			
		DISPLT	9.185E-03							
82.5	5.0	STRESS	-8.686E-03	-3.773E-01	4.699E-02	-1.881E-01	-3.390E-01	0.19	-0.11	-1.67
		STRAIN	1.370E-06	-5.891E-06	2.466E-06	-7.411E-06	-2.054E-06			
		DISPLT	9.185E-03							
82.5	-34.0	STRESS	-2.417E-01	1.594E-01	-5.329E-01	-2.843E-01	-6.152E-01	0.28	-0.21	-1.38
		STRAIN	-7.951E-06	2.403E-05	-3.118E-05	-4.535E-05	-1.510E-05			
		DISPLT	9.146E-03							
82.5	34.0	STRESS	-2.417E-01	-3.724E-02	-3.922E-01	-2.843E-01	-6.711E-01	0.15	-0.22	-0.65
		STRAIN	-7.767E-06	2.403E-05	-3.118E-05	-8.845E-05	-1.491E-05			
		DISPLT	9.146E-03							

THE PROBLEM PARAMETERS ARE

TOTAL LOAD.. 23750.00 LBS

TIRE PRESSURE.. 160.00 PSI

LOAD RADIUS.. 6.87 IN.

LAYER 1 HAS MODULUS 200000. POISSONS RATIO 0.500 AND THICKNESS 5.00 IN.
LAYER 2 HAS MODULUS 66000. POISSONS RATIO 0.300 AND THICKNESS 11.00 IN.
LAYER 3 HAS MODULUS 31000. POISSONS RATIO 0.300 AND THICKNESS 6.00 IN.
LAYER 4 HAS MODULUS 24500. POISSONS RATIO 0.300 AND THICKNESS 6.00 IN.
LAYER 5 HAS MODULUS 16300. POISSONS RATIO 0.300 AND THICKNESS 6.00 IN.
LAYER 6 HAS MODULUS 9000. POISSONS RATIO 0.400 AND IS SEMI-INFINITE.

TIRE	X-ORD	Y-ORD	RADIUS	CON.PRESS
1	-10.50	-23.00	6.87	160.00
2	10.50	-23.00	6.87	160.00
3	-10.50	23.00	6.87	160.00
4	10.50	23.00	6.87	160.00

X-ORDINATE = 0. Y-ORDINATE = 0.

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0	STRESS	0.17	-34.68	-49.23	0.00	0.00	-0.00	20.73	-27.91	-0.74
	STRAIN	2.106E-04	-5.077E-05	-1.599E-04	5.234E-13	1.108E-13	-8.986E-14			
	DISPLT	8.651E-02								
5.0	STRESS	0.17	-7.09	-12.62	0.00	-0.00	0.00	5.24	-6.51	-0.80
	STRAIN	9.222E-05	-5.077E-05	-1.599E-04	2.226E-14	-2.862E-13	4.002E-13			
	DISPLT	8.651E-02								
-34.0	STRESS	-8.60	5.72	2.64	-0.00	0.00	-0.00	6.15	-0.08	-75.82
	STRAIN	-6.812E-04	4.605E-04	2.147E-04	-2.343E-13	1.072E-13	-1.093E-12			
	DISPLT	7.687E-02								
34.0	STRESS	-8.60	0.12	-1.46	0.00	-0.00	-0.00	3.79	-3.31	-1.15
	STRAIN	-8.959E-04	4.605E-04	2.147E-04	3.313E-13	-5.572E-13	-1.713E-12			
	DISPLT	7.687E-02								

X-ORDINATE = 3.50 Y-ORDINATE = 0.

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0	STRESS	-0.29	-34.00	-49.41	0.00	-1.38	-0.00	20.51	-27.90	-0.74
	STRAIN	2.071E-04	-4.574E-05	-1.613E-04	9.012E-14	-2.077E-05	-8.203E-13			
	DISPLT	8.630E-02								
5.0	STRESS	-0.29	-6.95	-12.82	-0.00	-1.38	0.00	5.12	-6.69	-0.77
	STRAIN	8.552E-05	-4.574E-05	-1.613E-04	-1.324E-13	-5.454E-05	3.971E-13			
	DISPLT	8.630E-02								

-34.0STRESS	-8.54	5.63	2.62	0.00	-0.49	-0.00	6.09	-0.10	-63.55
STRAIN	-6.754E-04	4.542E-04	2.142E+04	1.125E-13	-7.830E-05	-6.362E-13			
DISPLT	7.665E-02								
34.0STRESS	-8.54	0.09	-1.45	-0.00	-0.49	-0.00	3.76	-3.30	-1.14
STRAIN	-8.882E-04	4.542E-04	2.142E+04	-8.898E-14	-1.527E-04	-5.877E-12			
DISPLT	7.665E-02								

X-ORDINATE = 7.00 Y-ORDINATE = 0.

DEPTH	ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0STRESS	-0.72	-32.98	-49.85	-0.00	-2.99	-0.00	20.38	-27.85	-0.73
STRAIN	2.035E-04	-3.849E-05	-1.650E+04	-3.999E-13	-4.492E-05	-6.838E-13			
DISPLT	8.566E-02								
5.0STRESS	-0.72	-6.69	-13.11	-0.00	-2.99	0.00	5.06	-6.84	-0.74
STRAIN	7.909E-05	-3.849E-05	-1.650E+04	-1.828E-13	-1.180E-04	6.055E-13			
DISPLT	8.566E-02								
-34.0STRESS	-8.40	5.42	2.64	-0.00	-0.95	0.00	5.97	-0.11	-52.51
STRAIN	-6.640E-04	4.384E-04	2.172E+04	-2.923E-13	-1.520E-04	8.204E-13			
DISPLT	7.605E-02								
34.0STRESS	-8.40	0.03	-1.40	-0.00	-0.95	0.00	3.68	-3.26	-1.13
STRAIN	-8.728E-04	4.384E-04	2.172E+04	-2.500E-13	-2.965E-04	1.600E-12			
DISPLT	7.605E-02								

X-ORDINATE = 10.50 Y-ORDINATE = 0.

DEPTH	ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0STRESS	-0.77	-32.03	-48.83	0.00	-4.61	0.00	19.91	-27.21	-0.73
STRAIN	1.983E-04	-3.618E-05	-1.621E+04	6.854E-14	-6.921E-05	8.941E-13			
DISPLT	8.445E-02								
5.0STRESS	-0.77	-6.48	-12.88	0.00	-4.61	0.	4.94	-6.71	-0.74
STRAIN	7.631E-05	-3.618E-05	-1.621E+04	1.699E-13	-1.818E-04	0.			
DISPLT	8.445E-02								
-34.0STRESS	-8.18	5.07	2.66	-0.00	-1.38	0.	5.76	-0.15	-38.40
STRAIN	-6.443E-04	4.130E-04	2.202E+04	-2.412E-13	-2.202E-04	0.			
DISPLT	7.509E-02								
34.0STRESS	-8.18	-0.09	-1.33	0.00	-1.38	0.00	3.56	-3.20	-1.11
STRAIN	-8.464E-04	4.130E-04	2.202E+04	2.480E-13	-4.296E-04	4.636E-12			
DISPLT	7.509E-02								

X-ORDINATE = 0. Y-ORDINATE = 4.60

DEPTH	ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0STRESS	-1.84	-36.50	-53.39	-0.00	0.00	4.51	21.46	-30.58	-0.70
STRAIN	2.155E-04	-4.444E-05	-1.711E+04	-3.449E-13	1.784E-13	6.768E-05			
DISPLT	8.731E-02								
5.0STRESS	-1.84	-7.73	-14.16	0.00	0.00	4.51	5.03	-7.91	-0.64
STRAIN	7.166E-05	-4.444E-05	-1.711E+04	8.261E-14	4.583E-13	1.778E-04			
DISPLT	8.731E-02								

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCT SHEAR	OCT STRESS	RATIO
-34.0	STRESS	-8.70	5.78	2.89	-0.00	0.00	0.01	6.26	-0.01	-590.83
	STRAIN	-6.933E-04	4.614E-04	2.312E+04	-1.418E-12	3.587E-13	1.372E-06			
	DISPLT	7.674E-02								
34.0	STRESS	-8.70	0.13	-1.35	-0.00	0.00	0.01	3.86	-3.30	-1.17
	STRAIN	-9.128E-04	4.614E-04	2.312E+04	-3.253E-13	6.997E-13	2.676E-06			
	DISPLT	7.674E-02								

X-ORDINATE = 3.50 Y-ORDINATE = 4.60

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCT SHEAR	OCT STRESS	RATIO
-5.0	STRESS	-1.98	-35.49	-53.74	-0.90	-1.04	4.71	21.44	-30.40	-0.71
	STRAIN	2.132E-04	-3.816E-05	-1.751E+04	-1.348E-05	-1.558E-05	7.069E-05			
	DISPLT	8.706E-02								
5.0	STRESS	-1.98	-7.42	-14.37	-0.38	-1.04	4.71	5.07	-7.92	-0.64
	STRAIN	6.914E-05	-3.816E-05	-1.751E+04	-1.480E-05	-4.091E-05	1.857E-04			
	DISPLT	8.706E-02								
-34.0	STRESS	-8.64	5.70	2.88	0.02	-0.49	0.01	6.20	-0.02	-263.56
	STRAIN	-6.882E-04	4.557E-04	2.307E+04	2.723E-06	-7.839E-05	9.143E-07			
	DISPLT	7.654E-02								
34.0	STRESS	-8.64	0.11	-1.34	0.00	-0.49	0.01	3.83	-3.29	-1.16
	STRAIN	-9.059E-04	4.557E-04	2.307E+04	1.318E-06	-1.529E-04	1.783E-06			
	DISPLT	7.654E-02								

X-ORDINATE = 7.00 Y-ORDINATE = 4.60

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCT SHEAR	OCT STRESS	RATIO
-5.0	STRESS	-2.37	-33.17	-54.07	-0.24	-2.57	5.14	21.24	-29.87	-0.71
	STRAIN	2.063E-04	-2.476E-05	-1.815E+04	-3.645E-06	-3.856E-05	7.714E-05			
	DISPLT	8.627E-02								
5.0	STRESS	-2.37	-6.76	-14.72	-0.23	-2.57	5.14	5.11	-7.95	-0.64
	STRAIN	6.175E-05	-2.476E-05	-1.815E+04	-8.889E-06	-1.013E-04	2.026E-04			
	DISPLT	6.627E-02								
-34.0	STRESS	-8.48	5.46	2.83	-0.08	-0.96	-0.00	6.05	-0.06	-96.65
	STRAIN	-6.728E-04	4.386E-04	2.295E+04	-1.314E-05	-1.539E-04	-5.802E-07			
	DISPLT	7.595E-02								
34.0	STRESS	-8.48	0.03	-1.31	-0.01	-0.96	-0.00	3.74	-3.25	-1.15
	STRAIN	-8.851E-04	4.386E-04	2.295E+04	-1.926E-06	-3.002E-04	-1.132E-06			
	DISPLT	7.595E-02								

X-ORDINATE = 10.50 Y-ORDINATE = 4.60

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCT SHEAR	OCT STRESS	RATIO
-5.0	STRESS	-2.89	-31.22	-58.87	2.30	-4.71	5.40	20.47	-28.99	-0.71
	STRAIN	1.958E-04	-1.671E-05	-1.791E+04	3.445E-05	-7.070E-05	8.099E-05			
	DISPLT	8.498E-02								
5.0	STRESS	-2.89	-6.35	-14.59	0.56	-4.71	5.40	4.91	-7.94	-0.62
	STRAIN	5.143E-05	-1.671E-05	-1.791E+04	2.188E-05	-1.857E-04	2.127E-04			
	DISPLT	8.498E-02								

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-34.0	STRESS	-8.21	5.06	2.78	-0.32	-1.40	-0.02	5.79	-0.12	-47.00
	STRAIN	-6.480E-04	4.117E-04	2.283E-04	-5.102E-05	-2.230F-04	-3.516E-06			
	DISPLT	7.498E-02								
34.0	STRESS	-8.21	-0.10	-1.27	-0.04	-1.40	-0.02	3.58	-3.19	-1.12
	STRAIN	-8.517E-04	4.117E-04	2.283E-04	-1.141E-05	-4.350E-04	-6.861E-06			
	DISPLT	7.498E-02								

X-ORDINATE = 0. Y-ORDINATE = 9.20

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0	STRESS	-6.16	-43.78	-58.76	0.00	0.00	10.66	22.13	-36.23	-0.61
	STRAIN	2.256E-04	-5.661E-05	-1.689E-04	1.346E-12	9.208F-14	1.599E-04			
	DISPLT	8.900E-02								
5.0	STRESS	-6.16	-10.42	-16.12	-0.00	-0.00	10.66	4.08	-10.90	-0.37
	STRAIN	2.736E-05	-5.661E-05	-1.689E-04	-1.157E-12	-6.196E-13	4.199E-04			
	DISPLT	8.900E-02								
-34.0	STRESS	-8.92	5.89	3.46	0.00	0.00	-0.10	6.48	0.14	46.00
	STRAIN	-7.193E-04	4.617E-04	2.680E-04	7.319E-13	6.729E-13	-1.520E-05			
	DISPLT	7.645E-02								
34.0	STRESS	-8.92	0.15	-1.10	0.00	-0.00	-0.10	4.01	-3.29	-1.22
	STRAIN	-9.491E-04	4.617E-04	2.680E-04	4.314E-15	-1.006E-12	-2.965E-05			
	DISPLT	7.645E-02								

X-ORDINATE = 3.50 Y-ORDINATE = 9.20

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0	STRESS	-6.26	-41.29	-60.55	-3.22	-0.29	11.17	22.47	-36.03	-0.62
	STRAIN	2.233E-04	-3.944E-05	-1.839E-04	-4.828E-05	-4.290E-06	1.675E-04			
	DISPLT	8.874E-02								
5.0	STRESS	-6.26	-9.54	-16.88	-1.20	-0.29	11.17	4.44	-10.89	-0.41
	STRAIN	2.528E-05	-3.944E-05	-1.839E-04	-4.746E-05	-1.127E-05	4.400E-04			
	DISPLT	8.874E-02								
-34.0	STRESS	-8.87	5.81	3.44	0.17	-0.50	-0.09	6.43	0.13	50.25
	STRAIN	-7.143E-04	4.563E-04	2.675E-04	2.676E-05	-7.909E-05	-1.514E-05			
	DISPLT	7.626E-02								
34.0	STRESS	-8.87	0.12	-1.09	0.01	-0.50	-0.09	3.98	-3.28	-1.21
	STRAIN	-9.423E-04	4.563E-04	2.675E-04	3.763E-06	-1.543E-04	-2.952E-05			
	DISPLT	7.626E-02								

X-ORDINATE = 7.00 Y-ORDINATE = 9.20

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0	STRESS	-6.46	-35.59	-63.97	-2.00	-1.87	12.16	23.48	-35.34	-0.66
	STRAIN	2.166E-04	-1.901E-06	-2.147E-04	-3.007E-05	-2.799E-05	1.823E-04			
	DISPLT	8.793E-02								
5.0	STRESS	-6.46	-7.58	-18.38	-0.97	-1.87	12.16	5.38	-10.81	-0.50
	STRAIN	2.012E-05	-1.901E-06	-2.147E-04	-3.802E-05	-7.350E-05	4.789E-04			
	DISPLT	8.793E-02								

PAGE 8

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-34.0	STRESS	-8.7	5.57	3.39	-0.02	-0.98	-0.10	6.28	0.09	71.99
	STRAIN	-6.988E-04	4.395E-04	2.658E-04	-2.503E-06	-1.561E-04	-1.550E-05			
	DISPLT	7.569E-02								
34.0	STRESS	-6.7	0.05	-1.07	-0.02	-0.98	-0.10	3.89	-3.24	-1.20
	STRAIN	-9.214E-04	4.395E-04	2.658E-04	-5.515E-06	-3.044E-04	-3.023E-05			
	DISPLT	7.569E-02								

X-ORDINATE = 10.50 Y-ORDINATE = -9.20

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0	STRESS	-6.58	-31.07	-64.26	4.45	-5.01	12.40	23.54	-33.97	-0.70
	STRAIN	2.054E-04	2.175E-05	-2.272E-04	3.670E-05	-7.522E-05	1.860E-04			
	DISPLT	8.648E-02								
5.0	STRESS	-6.58	-6.19	-16.82	1.01	-5.01	12.40	5.87	-10.53	-0.56
	STRAIN	1.394E-05	2.175E-05	-2.272E-04	3.968E-05	-1.976E-04	4.884E-04			
	DISPLT	8.648E-02								
-34.0	STRESS	-8.42	5.16	3.30	-0.61	-1.43	-0.11	6.01	0.01	403.63
	STRAIN	-6.721E-04	4.1 9E-04	2.623E-04	-9.676E-05	-2.278E-04	-1.755E-05			
	DISPLT	7.473E-02								
34.0	STRESS	-8.42	-0.08	-1.04	-0.10	-1.43	-0.11	3.72	-3.18	-1.17
	STRAIN	-8.853E-04	4.1 9E-04	2.623E-04	-3.017E-05	-4.442E-04	-3.423E-05			
	DISPLT	7.473E-02								

X-ORDINATE = 0. Y-ORDINATE = 13.80

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0	STRESS	-12.43	-61.31	-55.86	-0.00	-0.00	15.09	21.87	-43.20	-0.51
	STRAIN	2.308E-04	-1.358E-04	-9.497E-05	-1.108E-13	-2.148E-14	2.263E-04			
	DISPLT	9.095E-02								
5.0	STRESS	-12.43	-17.24	-15.17	0.00	-0.00	15.09	1.97	-14.95	-0.13
	STRAIN	-4.094E-05	-1.358E-04	-9.497E-05	8.706E-13	-1.636E-14	5.943E-04			
	DISPLT	9.095E-02								
-34.0	STRESS	-9.13	5.98	4.07	-0.00	-0.00	-0.36	6.72	0.30	22.06
	STRAIN	-7.449E-04	4.598E-04	3.075E-04	-3.871E-13	-2.871E-13	-5.814E-05			
	DISPLT	7.594E-02								
34.0	STRESS	-9.13	0.16	-0.82	-0.00	-0.00	-0.36	4.17	-3.26	-1.28
	STRAIN	-9.849E-04	4.598E-04	3.075E-04	-2.448E-13	-5.600E-13	-1.134E-04			
	DISPLT	7.594E-02								

X-ORDINATE = 3.50 Y-ORDINATE = 13.80

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0	STRESS	-15.19	-54.66	-60.22	-9.23	3.53	18.62	20.04	-43.36	-0.46
	STRAIN	2.112E-04	-8.478E-05	-1.265E-04	-1.385E-04	5.293E-05	2.793E-04			
	DISPLT	9.097E-02								
5.0	STRESS	-15.19	-15.41	-17.53	-3.73	3.53	18.62	1.05	-16.04	-0.07
	STRAIN	-8.046E-05	-8.478E-05	-1.265E-04	-1.471E-04	1.390E-04	7.335E-04			
	DISPLT	9.097E-02								

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-34.0	STRESS	-9.06	5.89	4.04	0.69	-0.50	-0.37	6.66	0.29	22.65
	STRAIN	-7.386E-04	4.539E-04	3.063E-04	1.097E-04	-7.906E-05	-5.865E-05			
	DISPLT	7.567E-02								
34.0	STRESS	-9.06	0.14	-0.81	0.06	-0.50	-0.37	4.13	-3.24	-1.27
	STRAIN	-9.764E-04	4.539E-04	3.063E-04	2.021E-05	-1.542E-04	-1.144E-04			
	DISPLT	7.567E-02								

X-ORDINATE = 7.00 Y-ORDINATE = 13.80

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0	STRESS	-23.05	-31.68	-60.30	-2.67	1.89	23.81	15.92	-38.34	-0.42
	STRAIN	1.147E-04	4.994E-05	-1.647E-04	-4.011E-05	2.836E-05	3.571E-04			
	DISPLT	9.068E-02								
5.0	STRESS	-23.05	-9.84	-20.73	-3.19	1.89	23.81	5.76	-17.87	-0.32
	STRAIN	-2.102E-04	4.994E-05	-1.647E-04	-1.255E-04	7.449E-05	9.378E-04			
	DISPLT	9.068E-02								
-34.0	STRESS	-8.86	5.62	3.94	0.56	-0.99	-0.37	6.47	0.24	27.42
	STRAIN	-7.195E-04	4.356E-04	3.013E-04	8.976E-05	-1.574E-04	-5.902E-05			
	DISPLT	7.513E-02								
34.0	STRESS	-8.86	0.05	-0.81	0.02	-0.99	-0.37	4.01	-3.21	-1.25
	STRAIN	-9.505E-04	4.356E-04	3.013E-04	5.817E-06	-3.070E-04	-1.151E-04			
	DISPLT	7.513E-02								

X-ORDINATE = 10.50 Y-ORDINATE = 13.80

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0	STRESS	-26.91	-16.68	-58.27	4.99	-5.90	26.09	17.70	-33.95	-0.52
	STRAIN	5.280E-05	1.296E-04	-1.824E-04	7.482E-05	-8.856E-05	3.914E-04			
	DISPLT	8.938E-02								
5.0	STRESS	-26.91	-6.10	-21.94	1.17	-5.90	26.09	8.87	-18.32	-0.48
	STRAIN	-2.803E-04	1.296E-04	-1.824E-04	4.615E-05	-2.326E-04	1.028E-03			
	DISPLT	8.938E-02								
-34.0	STRESS	-8.55	5.20	3.79	-0.69	-1.45	-0.38	6.17	0.15	42.07
	STRAIN	-6.898E-04	4.063E-04	2.942E-04	-1.099E-04	-2.311E-04	-6.005E-05			
	DISPLT	7.420E-02								
34.0	STRESS	-8.55	-0.08	-0.80	-0.16	-1.45	-0.38	3.83	-3.14	-1.22
	STRAIN	-9.102E-04	4.063E-04	2.942E-04	-4.896E-05	-4.508E-04	-1.171E-04			
	DISPLT	7.420E-02								

X-ORDINATE = 0. Y-ORDINATE = 18.40

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0	STRESS	-24.14	-90.87	-33.18	-0.00	0.00	13.86	29.55	-49.40	-0.60
	STRAIN	1.894E-04	-3.110E-04	1.216E-04	-5.696E-13	1.744E-12	2.080E-04			
	DISPLT	9.269E-02								
5.0	STRESS	-24.14	-30.26	-8.29	0.00	0.00	13.86	9.26	-20.90	-0.44
	STRAIN	-1.906E-04	-3.110E-04	1.216E-04	9.253E-13	3.791E-12	5.461E-04			
	DISPLT	9.269E-02								

************************ ************************

34.0STRESS	-9.16	5.97	4.48	-0.00	-0.00	-0.82	6.81	0.43	15.84
STRAIN	-7.544E-04	4.526E-04	3.333E-04	-3.341E-13	-4.369E-13	-1.313E-04			
DISPLT	7.489E-02								
34.0STRESS	-9.16	0.17	-0.60	0.00	-0.00	-0.82	4.23	-3.20	-1.32
STRAIN	-9.991E-04	4.526E-04	3.333E-04	4.046E-13	-1.047E-12	-2.561E-04			
DISPLT	7.489E-02								

X-ORDINATE = 3.50 Y-ORDINATE = 18.40

DEPTH	ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-3.0STRESS	-39.38	-56.91	-18.33	6.59	12.15	17.40	15.77	-38.20	-0.41
STRAIN	-8.794E-05	-1.4[illegible]3E-04	1.491E-04	9.880E-05	1.822E-04	2.610E-04			
DISPLT	9.339E-02								
5.0STRESS	-39.38	-23.81	-9.12	-3.86	12.15	17.40	12.36	-24.10	-0.51
STRAIN	-4.470E-04	-1.4[illegible]3E-04	1.491E-04	-1.520E-04	4.785E-04	6.854E-04			
DISPLT	9.339E-02								
34.0STRESS	-9.06	5.81	4.37	1.21	-0.50	-0.82	6.70	0.38	17.84
STRAIN	-7.432E-04	4.430E-04	3.279E-04	1.934E-04	-7.938E-05	-1.312E-04			
DISPLT	7.485E-02								
34.0STRESS	-9.06	0.11	-8.63	0.09	-0.50	-0.82	4.16	-3.19	-1.30
STRAIN	-9.836E-04	4.430E-04	3.279E-04	2.692E-05	-1.546E-04	-2.560E-04			
DISPLT	7.485E-02								

X-ORDINATE = 7.00 Y-ORDINATE = 18.40

DEPTH	ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0STRESS	-67.33	1.03	9.03	37.13	9.59	24.06	34.26	-19.09	-1.79
STRAIN	-3.618E-04	1.5[illegible]9E-04	2.109E-04	5.569E-04	1.439E-04	3.609E-04			
DISPLT	9.441E-02								
5.0STRESS	-67.33	-13.32	-10.28	-3.07	9.59	24.06	26.21	-30.31	-0.86
STRAIN	-9.128E-04	1.5[illegible]9E-04	2.109E-04	-1.211E-04	3.779E-04	9.477E-04			
DISPLT	9.441E-02								
34.0STRESS	-8.84	5.51	4.21	1.87	-0.99	-0.82	6.48	0.29	22.03
STRAIN	-7.213E-04	4.230E-04	3.199E-04	2.985E-04	-1.574E-04	-1.308E-04			
DISPLT	7.441E-02								
34.0STRESS	-8.84	0.01	-8.65	0.10	-0.99	-0.82	4.02	-3.16	-1.27
STRAIN	-9.537E-04	4.230E-04	3.199E-04	3.078E-05	-3.069E-04	-2.552E-04			
DISPLT	7.441E-02								

X-ORDINATE = 10.50 Y-ORDINATE = 18.40

DEPTH	ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0STRESS	-78.06	27.93	21.58	2.62	-6.86	25.39	48.54	-9.52	-5.10
STRAIN	-5.141E-04	2.8[illegible]6E-04	2.333E-04	3.934E-05	-1.029E-04	3.808E-04			
DISPLT	9.344E-02								
5.0STRESS	-78.06	-8.01	-10.43	0.64	-6.86	25.39	32.47	-32.17	-1.01
STRAIN	-1.099E-03	2.8[illegible]8E-04	2.333E-04	2.531E-05	-2.701E-04	1.000E-03			
DISPLT	9.344E-02								

PAGE 11

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-34.0	STRESS	-8.53	5.09	4.05	-0.46	-1.45	-0.81	6.19	0.20	30.39
	STRAIN	-6.913E-04	3.948E-04	3.115E-04	-7.392E-05	-2.313E-04	-1.293E-04			
	DISPLT	7.347E-02								
34.0	STRESS	-8.53	-0.12	-0.66	-0.20	-1.45	-0.81	3.84	-3.10	-1.24
	STRAIN	-9.129E-04	3.948E-04	3.115E-04	-6.313E-05	-4.511E-04	-2.523E-04			
	DISPLT	7.347E-02								

X-ORDINATE = 0. Y-ORDINATE = 23.00

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0	STRESS	-34.25	-96.52	-7.91	-0.00	0.	-2.84	38.06	-46.90	-0.81
	STRAIN	9.480E-05	-3.872E-04	2.924E-04	-1.020E-13	0.	-4.253E-05			
	DISPLT	9.233E-02								
5.0	STRESS	-34.25	-36.40	-1.90	0.00	0.	-2.84	15.78	-24.19	-0.65
	STRAIN	-3.449E-04	-3.872E-04	2.924E-04	2.309E-14	0.	-1.117E-04			
	DISPLT	9.233E-02								
-34.0	STRESS	-8.93	5.78	4.45	-0.00	0.	-1.40	6.64	0.43	15.33
	STRAIN	-7.361E-04	4.371E-04	3.310E-04	-1.993E-14	0.	-2.236E-04			
	DISPLT	7.344E-02								
34.0	STRESS	-8.93	0.15	-0.53	0.00	0.	-1.40	4.13	-3.11	-1.33
	STRAIN	-9.750E-04	4.371E-04	3.310E-04	1.907E-13	0.	-4.360E-04			
	DISPLT	7.344E-02								

X-ORDINATE = 3.50 Y-ORDINATE = 23.00

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0	STRESS	-55.19	-52.76	14.83	-0.82	19.18	-2.81	32.45	-31.04	-1.05
	STRAIN	-1.811E-04	-1.629E-04	3.440E-04	-1.232E-05	2.677E-04	-4.215E-05			
	DISPLT	9.344E-02								
5.0	STRESS	-55.19	-27.98	-2.25	-0.20	19.18	-2.81	21.62	-28.47	-0.76
	STRAIN	-6.988E-04	-1.629E-04	3.440E-04	-7.820E-06	7.556E-04	-1.107E-04			
	DISPLT	9.344E-02								
-34.0	STRESS	-8.83	5.60	4.32	0.01	-0.49	-1.39	6.52	0.36	17.87
	STRAIN	-7.241E-04	4.266E-04	3.243E-04	1.504E-06	-7.824E-05	-2.225E-04			
	DISPLT	7.352E-02								
34.0	STRESS	-8.83	0.08	-0.58	-0.08	-0.49	-1.39	4.05	-3.11	-1.30
	STRAIN	-9.583E-04	4.266E-04	3.243E-04	-2.476E-05	-1.526E-04	-4.339E-04			
	DISPLT	7.352E-02								

X-ORDINATE = 7.00 Y-ORDINATE = 23.00

DEPTH		ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0	STRESS	-89.23	18.79	46.53	-1.60	12.02	-2.74	58.56	-7.97	-7.35
	STRAIN	-6.094E-04	2.007E-04	4.088E-04	-2.403E-05	1.803E-04	-4.103E-05			
	DISPLT	9.475E-02								
5.0	STRESS	-89.23	-14.79	-4.23	-0.39	12.02	-2.74	37.83	-36.08	-1.05
	STRAIN	-1.265E-03	2.007E-04	4.088E-04	-1.526E-05	4.735E-04	-1.078E-04			
	DISPLT	9.475E-02								

PAGE 12

-34.0 STRESS	-8.62	5.34	4.19	0.02	-0.96	-1.37	6.33	0.30	20.94
STRAIN	-7.046E-04	4.090E-04	3.178E-04	2.546E-06	-1.537E-04	-2.193E-04			
DISPLT	7.297E-02								
34.0 STRESS	-8.62	-0.01	-0.59	-0.16	-0.96	-1.37	3.93	-3.07	-1.28
STRAIN	-9.318E-04	4.090E-04	3.178E-04	-4.896E-05	-2.998E-04	-4.277E-04			
DISPLT	7.297E-02								

X-ORDINATE = 10.50 Y-ORDINATE = 23.00

DEPTH	ZZ	XX	YY	XY	XZ	YZ	OCTSHEAR	OCTSTRESS	RATIO
-5.0 STRESS	-99.01	42.35	53.95	-2.31	-7.17	-2.62	89.53	-0.91	-76.65
STRAIN	-7.358E-04	3.244E-04	4.114E-04	-3.459E-05	-1.075E-04	-3.923E-05			
DISPLT	9.388E-02								
5.0 STRESS	-99.01	-9.96	-5.54	-0.56	-7.17	-2.62	43.06	-38.17	-1.13
STRAIN	-1.430E-03	3.244E-04	4.114E-04	-2.196E-05	-2.823E-04	-1.030E-04			
DISPLT	9.388E-02								
-34.0 STRESS	-8.31	4.93	4.01	0.02	-1.42	-1.34	6.04	0.21	28.73
STRAIN	-6.746E-04	3.816E-04	3.085E-04	2.688E-06	-2.262E-04	-2.141E-04			
DISPLT	7.205E-02								
34.0 STRESS	-8.31	-0.13	-0.60	-0.23	-1.42	-1.34	3.75	-3.02	-1.24
STRAIN	-8.911E-04	3.816E-04	3.085E-04	-7.210E-05	-4.412E-04	-4.176E-04			
DISPLT	7.205E-02								

THIS IS THE END

APPENDIX K: EXAMPLE DESIGN PROBLEMS

1. This appendix presents four example design problems which illustrate use of the design procedure described in the main text of this report. The examples are for a conventional flexible pavement, an all-bituminous concrete (ABC) pavement, a conventional flexible pavement in which the subgrade is stabilized with lime in accordance with Federal Aviation Administration Item P-155 (from Advisory Circular AC 150/5370-10[16]), and a chemically stabilized pavement in which the base and subbase courses are both stabilized with portland cement in accordance with Item P-304.

2. The four alternative designs are applied to a pavement for Shreveport, La., for 200,000 departures of a 750,000-lb B-747 aircraft over a 20-yr design life. The initial designs are for critical areas; appropriate thickness reduction factors will be applied for noncritical areas. Since the pavement will be located at Shreveport, frost design is not applicable.

3. The subgrade on which the pavement is to be constructed is a lean clay classified E-8, CL. Borrow materials available for construction are pit-run sand and gravel meeting Item P-154 and crushed limestone meeting Item P-209. The bituminous concrete consists of a 5.2 percent bitumen (85-100 penetration) and crushed limestone mixture meeting Item P-401.

PROBLEM 1: CONVENTIONAL FLEXIBLE PAVEMENT

4. To design a conventional flexible pavement, the performance model illustrated in Figure K1 is used. Input parameters required are:

- a. Bituminous concrete modulus for each month.
- b. Subgrade modulus.
- c. Traffic parameters.
- d. Limiting strain criteria.
- e. Estimated initial thickness.
- f. Base course modulus.
- g. Subbase course modulus.

Once these parameters have been determined, they are input to the performance model, and the iterative process is followed until an optimum pavement section has been generated.

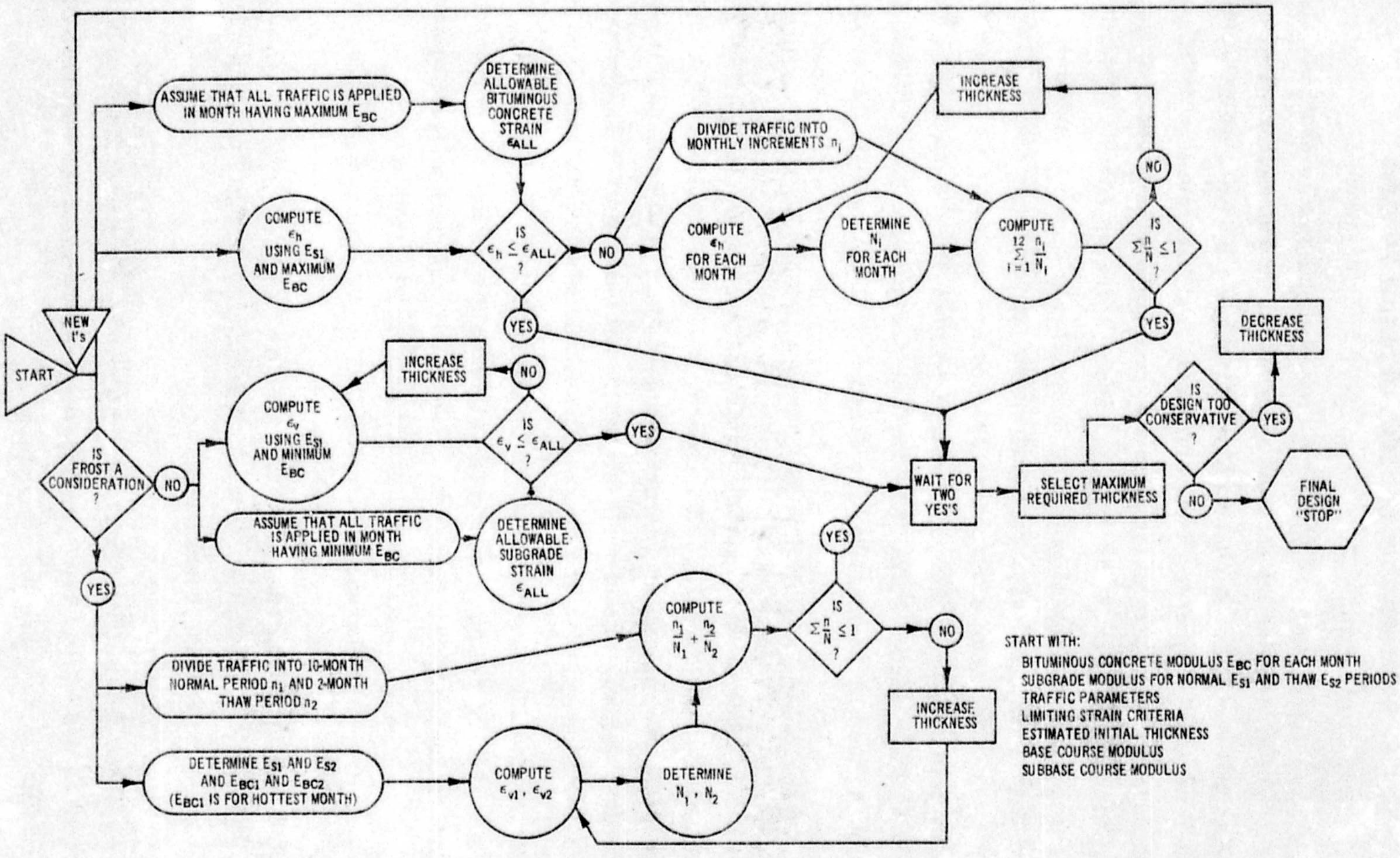


Figure K1. Flow diagram for conventional flexible pavement

TRAFFIC PARAMETERS

5. The design aircraft is a B-747 with a gross weight of 750,000-lb. Characteristics of this aircraft and other traffic data required in design are as follows:

- a. Wheel spacing: 44 by 58 in.
- b. Number of wheels in assembly: 4.
- c. Wheel load: 44,531 lb.
- d. Tire contact pressure: 182 psi.
- e. Design life: 20 yr.
- f. Design traffic: 10,000 annual departures (200,000 total departures over 20-yr design life).
- g. Factor for converting departures to coverages: 1.85.
- h. Total number of coverages: 108,108.

ESTIMATED INITIAL THICKNESS

6. The thickness design curves shown in Figure K2 are entered with the 750,000-lb gross aircraft weight of the B-747. A total pavement thickness of 39 in. and a required base course thickness of 12 in. are indicated. Estimated initial thicknesses are therefore 5 in. of bituminous concrete surface course (required for critical areas), 12 in. of base course, and 22 in. of subbase course.

BITUMINOUS CONCRETE MODULUS

7. To determine the modulus of the bituminous concrete surface course for each month, laboratory tests are first conducted on prepared specimens in accordance with the procedures outlined in Appendix E. The results of these tests are used to develop the relationship shown in Figure K3. Local climatological data are then used to estimate the design pavement temperature for each month for design based on bituminous concrete strain (Table K1) and subgrade strain (Table K2). These values can then be used to enter Figure K3 to determine the bituminous concrete modulus for each month.

SUBGRADE MODULUS

8. Since frost design is unnecessary for the Shreveport area, the subgrade modulus is determined only for the normal period. As the first step, the stress-modulus relationship for the subgrade is determined from

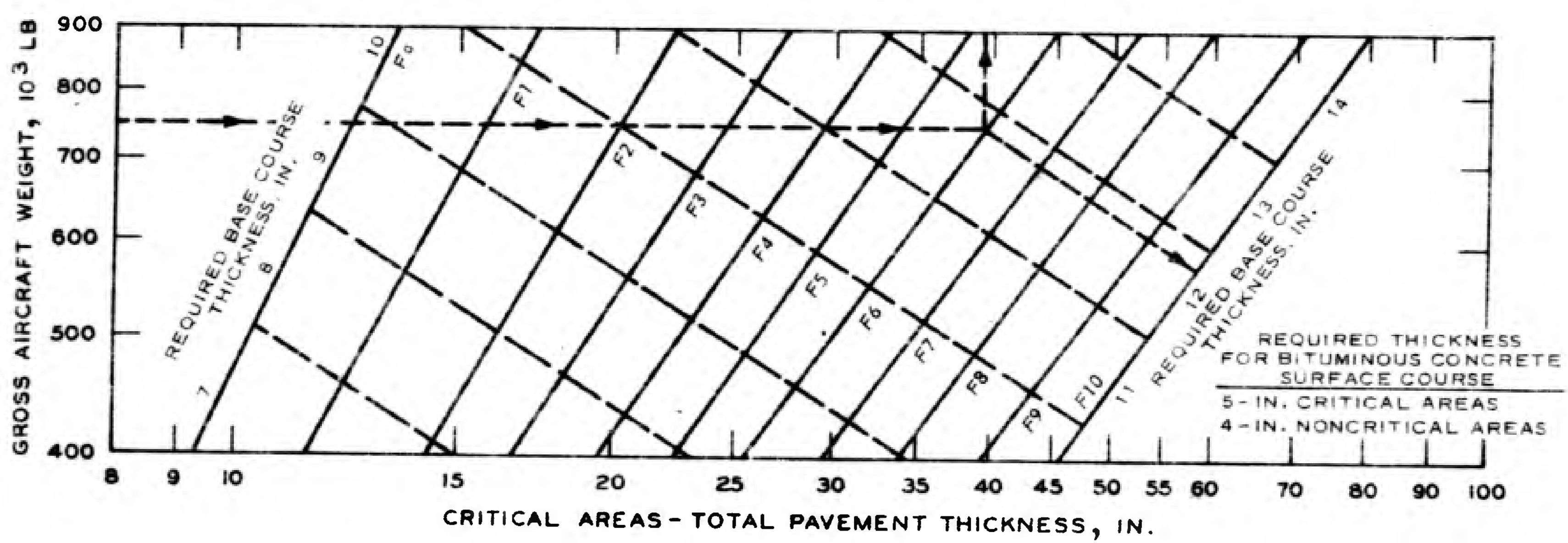


Figure K2. Flexible pavement design curves for B-747 (from AC 150/5320-6B[9])

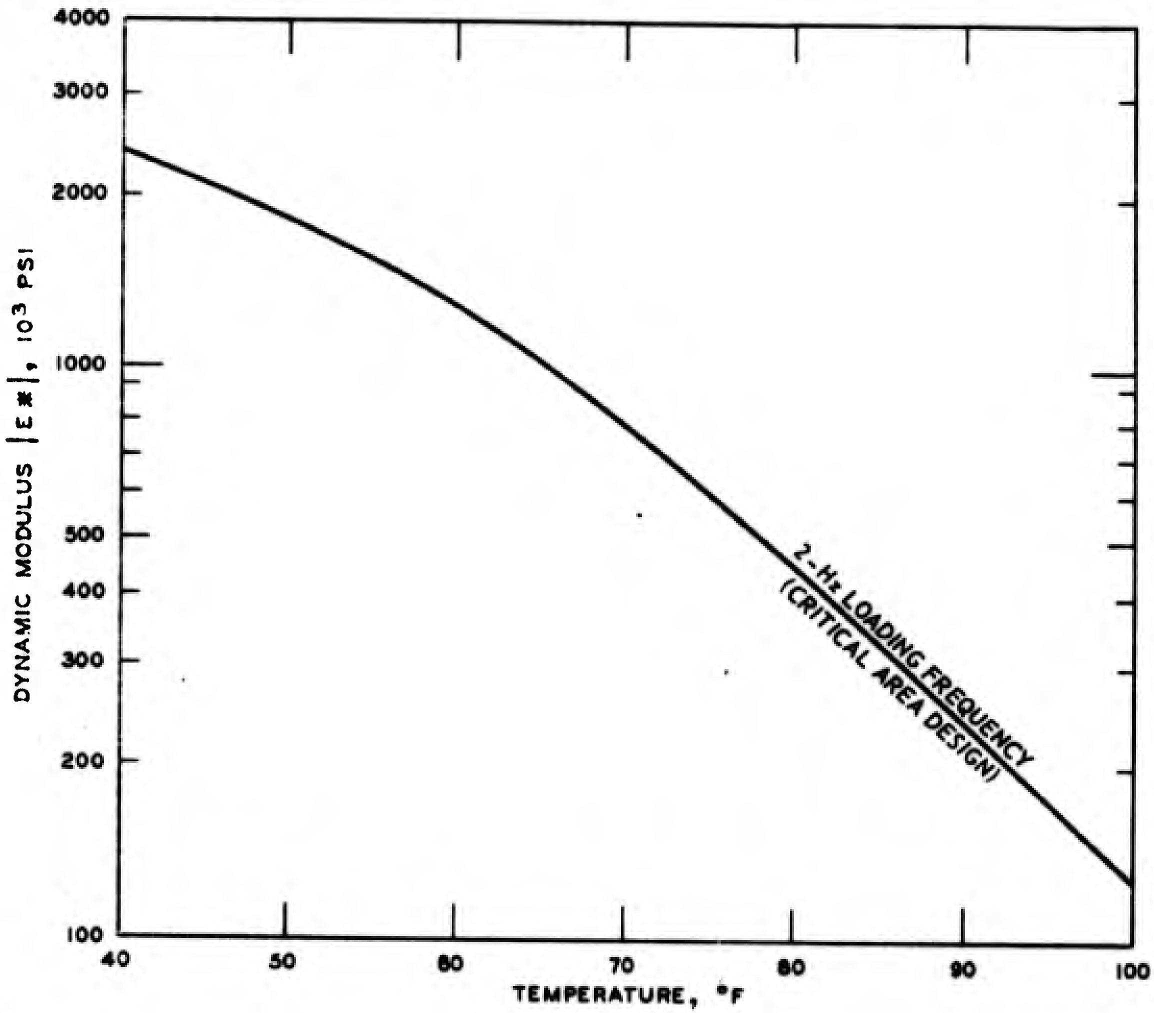


Figure K3. Results of laboratory tests to determine dynamic modulus of bituminous concrete

Table K1

Bituminous Concrete Moduli for Each Month For Conventional Flexible Pavement Design Based on Bituminous Concrete Strain

Month	Average Daily Mean Air Temperature* °F	Design Pavement Temperature** °F	Dynamic Modulus† |E*| 10^3 psi
Jan	47.5	56	1500
Feb	50.7	60	1270
Mar	58.0	67	920
Apr	66.1	76	570
May	73.3	84	360
Jun	80.5	92	220
Jul	83.1	95	180
Aug	82.7	95	180
Sep	77.3	89	260
Oct	67.2	77	540
Nov	56.2	65	1000
Dec	49.3	57	1400

* Determined from local climatological data for Shreveport, La. (such as presented in Appendix A).

** Estimated from 5-in. bituminous concrete thickness curve in Figure 2 of the main text. (In design for bituminous concrete strain, the average daily mean air temperature is used as the design air temperature for entering Figure 2.)

† Determined by entering Figure K3 with the design pavement temperature.

Table K2

Bituminous Concrete Moduli for Each Month
For Conventional Flexible Pavement Design
Based on Subgrade Strain

Month (1)	Average Daily Mean Air Temperature* °F (2)	Average Daily Maximum Air Temperature* °F (3)	Design Air Temperature** °F (4)	Design Pavement Temperature† °F (5)	Dynamic Modulus†† |E*| 10^3 psi (6)
Jan	47.5	56.4	52	60	1270
Feb	50.7	60.1	55	64	1060
Mar	58.0	68.0	63	72	700
Apr	66.1	76.0	71	81	420
May	73.3	83.2	78	90	250
Jun	80.5	90.4	85	97	160
Jul	83.1	92.9	88	100	130
Aug	82.7	92.8	88	100	130
Sep	77.3	87.4	82	94	190
Oct	67.2	78.1	73	83	380
Nov	56.2	66.4	61	71	720
Dec	49.3	58.3	54	61	1200

* Determined from local climatological data for Shreveport, La. (such as presented in Appendix A).
** Average of values from Columns 2 and 3.
† Estimated from 5-in. bituminous concrete thickness curve in Figure 2 of the main text. (Figure 2 is entered with the appropriate design air temperature.)
†† Determined by entering Figure K3 with the design pavement temperature.

laboratory tests conducted in accordance with the procedures outlined in Appendix C. Results of these laboratory tests are shown in Figure K4. To determine the design subgrade modulus, the resilient modulus curve from Figure K4 and a 10,000 annual departures curve interpolated from Figure C5 are plotted together as shown in Figure K5. The design subgrade modulus is the value obtained at the intersection of these two curves, i.e., 9000 psi. Since Poisson's ratio is also measured in these laboratory tests, the measured value of 0.45 is used in design.

BASE AND SUBBASE COURSE MODULI

9. Modulus values for the base and subbase course materials are determined in accordance with the procedures outlined in Appendix G. Figure G1 is entered with the modulus already determined for the subgrade of 9000 psi (modulus of layer n + 1), and the modulus of the lower 8 in. of the subbase course is read. This procedure is repeated for the other sublayers in the base and subbase courses as described in Appendix G. The resulting conventional flexible pavement section generated from the available input is shown in Figure K6.

LIMITING STRAIN CRITERIA

10. The criteria for limiting horizontal tensile strain in the bituminous concrete are shown in Figure K7, and those for limiting vertical compressive strain at the top of the subgrade are shown in Figure K8.

ANALYSIS OF DESIGN SECTION

11. An analysis of the initial design section (Figure K6) must now be made. The parameters outlined in the preceding paragraphs are first input to the performance model (Figure K1). Since frost design is not applicable to this pavement, the first step in the process is a comparison of the computed and allowable subgrade strains. The CHEVIT computer program is used to determine the computed subgrade strain based on the minimum bituminous concrete modulus of 130,000 psi (from Table K2) and the normal period subgrade modulus of 9000 psi. The computed subgrade strain ε_v is 9.2×10^{-3} in./in. To determine the allowable subgrade strain, Figure K8 is entered with the number of annual strain repetitions. For consideration of subgrade strain for critical traffic areas, 1 departure

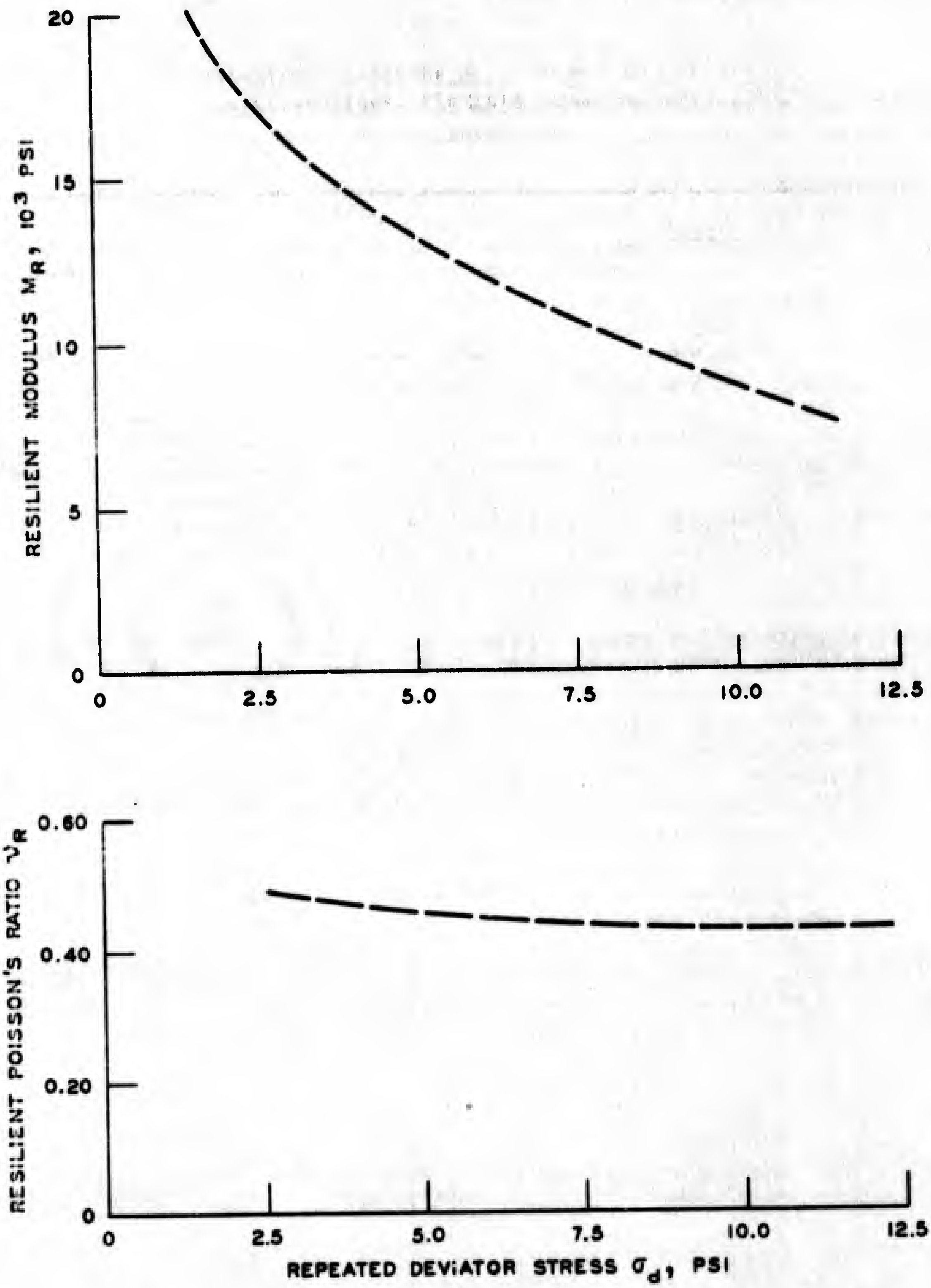


Figure K4. Results of laboratory tests to determine resilient modulus and resilient Poisson's ratio of lean clay (E-8, CL) subgrade soil

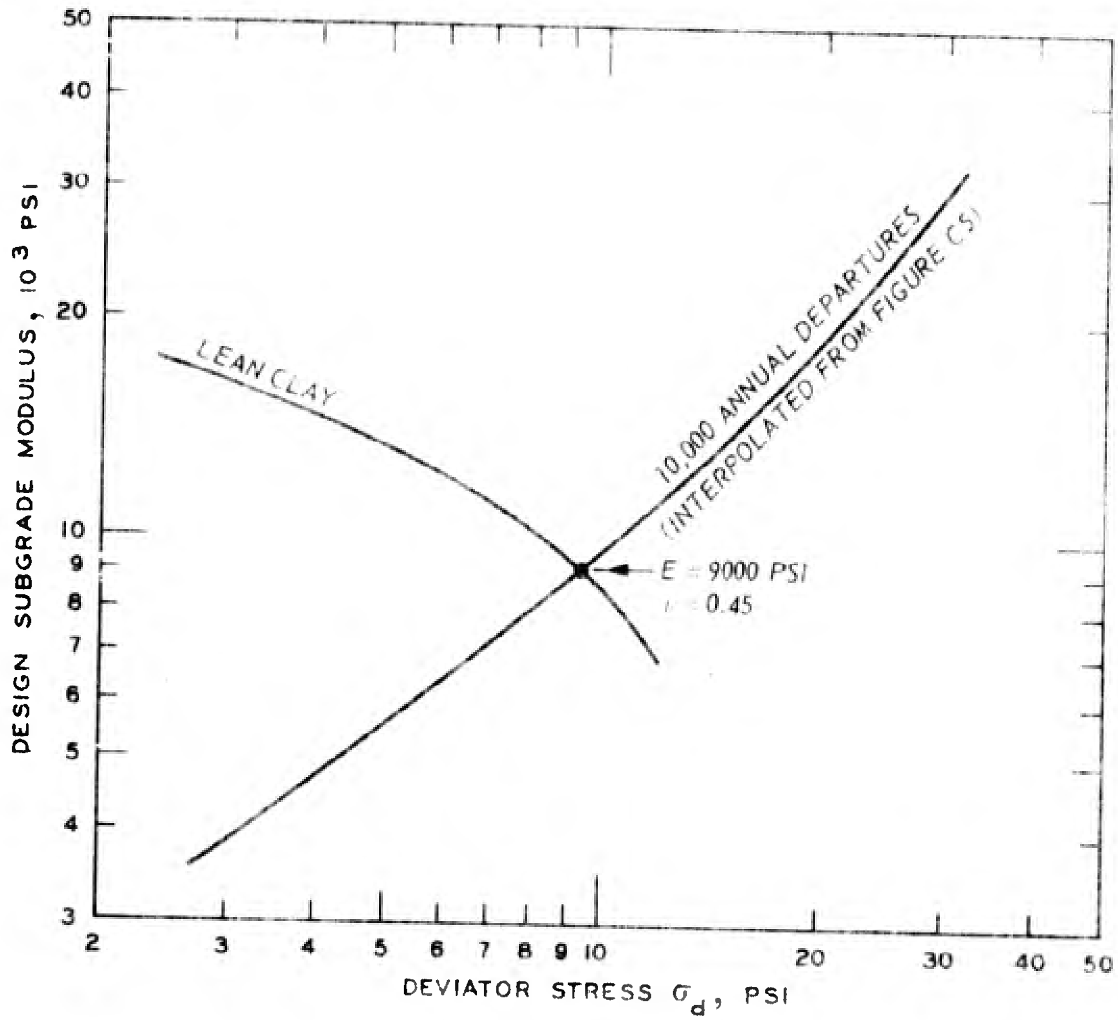


Figure K5. Determination of design subgrade modulus

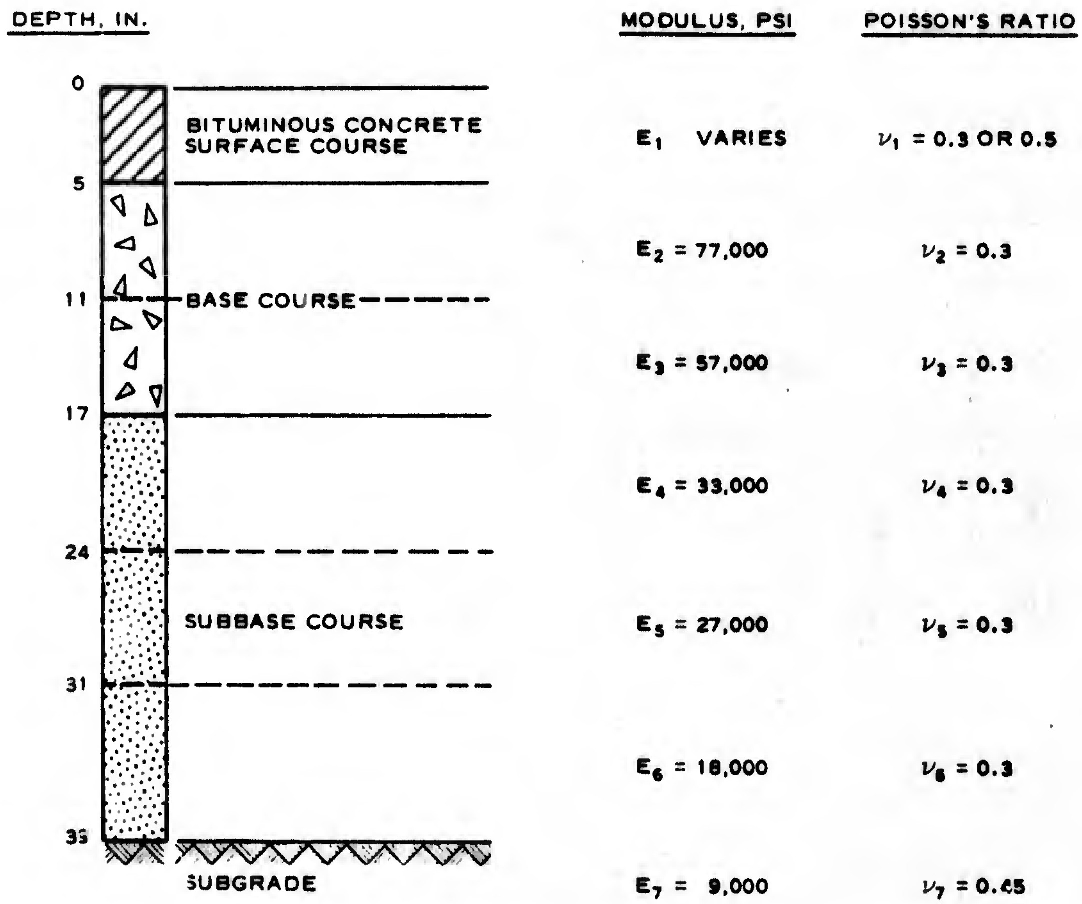


Figure K6. Estimated conventional flexible pavement section

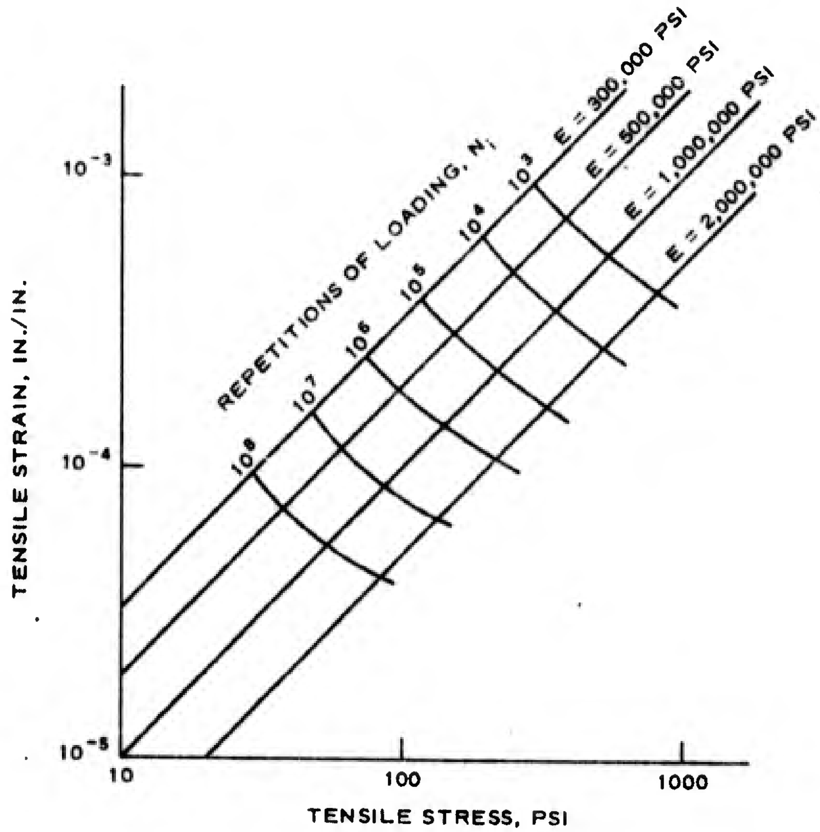


Figure K7. Criteria for limiting horizontal tensile strain in the bituminous concrete (after Heukelom and Klomp[30])

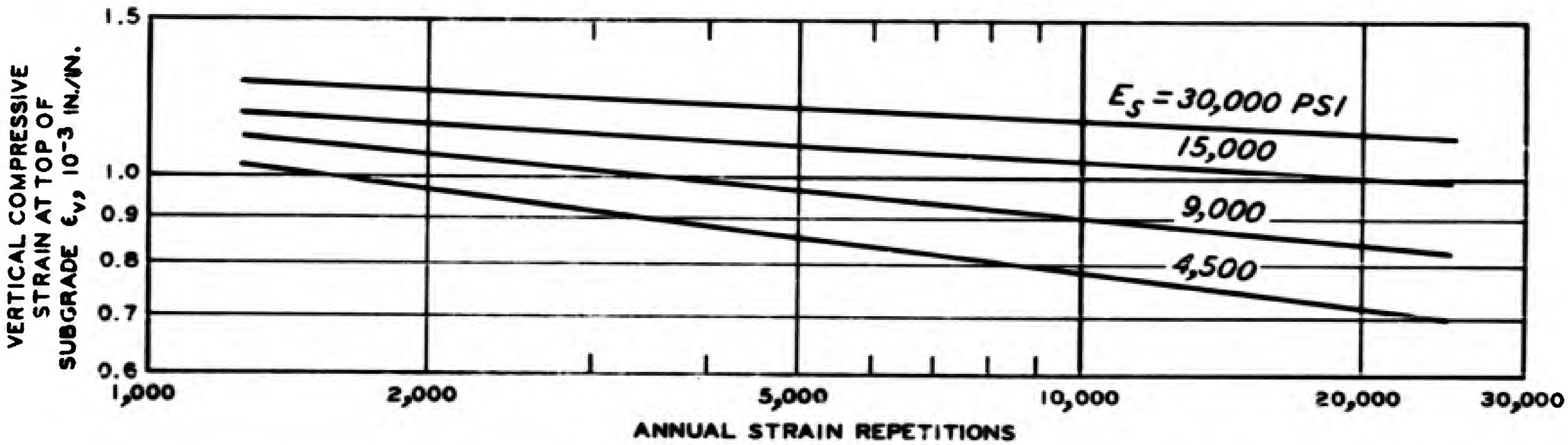


Figure K8. Criteria for limiting vertical compressive strain at the top of the subgrade results in the application of 1 strain repetition. Thus, for the design traffic level of 10,000 annual departures, Figure K8 is entered with 10,000 annual strain repetitions. The allowable strain ϵ_{ALL} is 9.4×10^{-3} in./in. Since the computed strain is less than the allowable strain, the design pavement section provides adequate protection for the subgrade. In addition, since the two strains are nearly equal, the design is not overly conservative with respect to limiting subgrade strain.

12. However, to illustrate how a more exact value for the required thickness can be determined with consideration to limiting subgrade strain, the two additional pavement sections shown in Figures K9 and K10 are included in the analysis. The computed subgrade strain for the 36-in. section is 10.5×10^{-3} in./in. and for the 45-in. section is 8.2×10^{-3} in./in. These two values are plotted along with the computed subgrade strain for the 39-in. section to develop the relationship between subgrade strain and pavement thickness shown in Figure K11. Entering this figure with the allowable subgrade strain of 9.4×10^{-3} in./in., a better estimate of the minimum thickness which meets the subgrade strain criteria is obtained. The estimated value of 38.5 in. rounded to the next higher inch results in a design thickness of 39 in.

13. The second step in the analysis of the design section involves checking for fatigue cracking of the bituminous concrete surface course. To begin this step, the most critical condition for the surface course is assumed; i.e., it is assumed that all of the traffic for the 20-yr design life of the pavement is applied during the single month for which the bituminous concrete modulus is at its maximum. Next, the computed and allowable horizontal tensile strains in the the surface course are determined. If the allowable strain is less than or equal to the computed strain under these extreme conditions, then the design is acceptable. However, if the computed strain is greater than the allowable strain, then the cumulative damage for the 20-yr design life must be determined.

14. To determine the allowable strain in the bituminous concrete, Figure K7 is entered with the modulus for January (1,500,000 psi from Table K1), which is the month for which this value is at its maximum, and the total number of strain repetitions. For consideration of bituminous

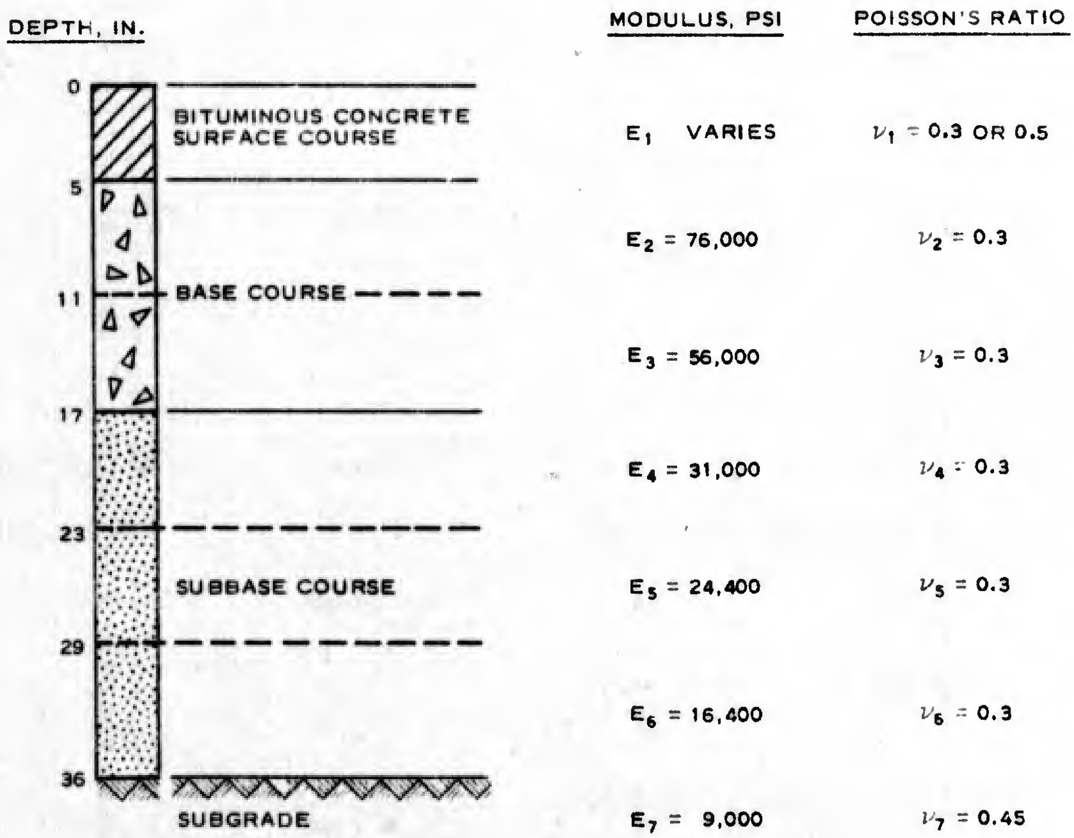


Figure K9. Estimated 36-in. conventional flexible pavement section

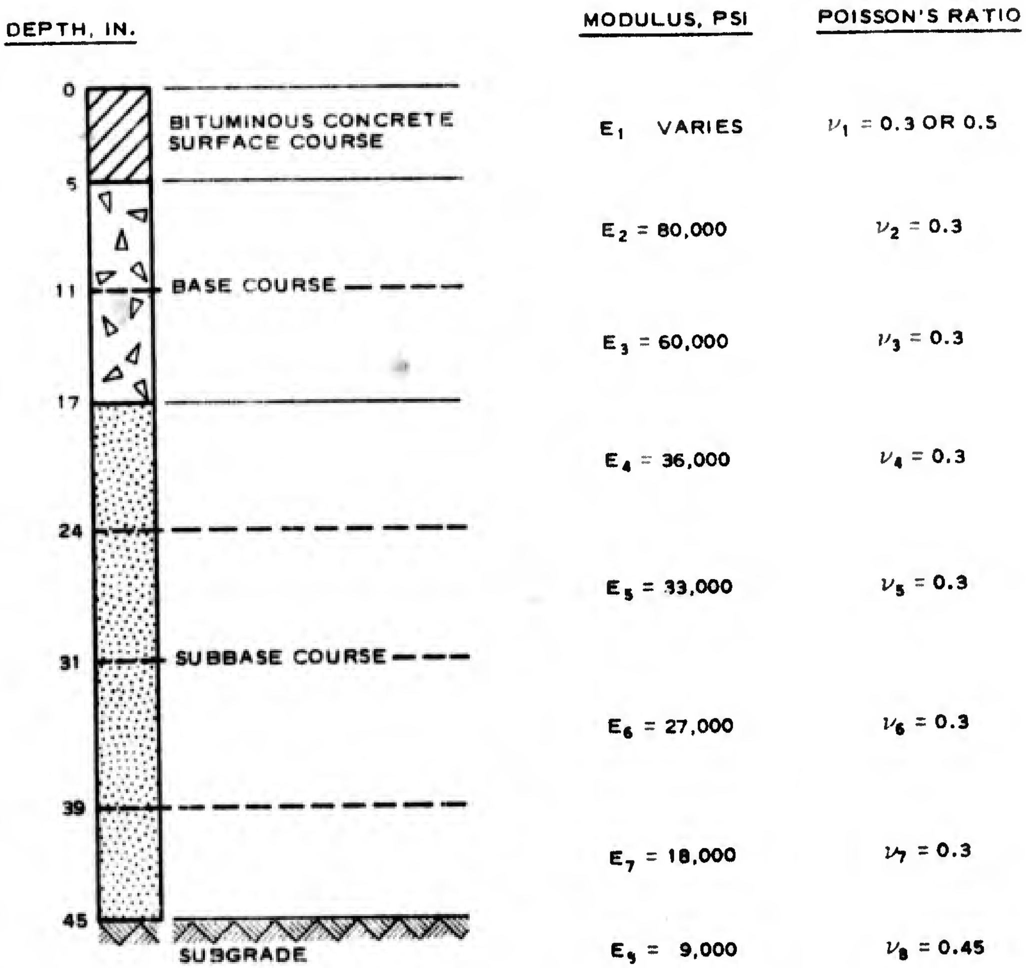


Figure K10. Estimated 45-in. conventional flexible pavement section

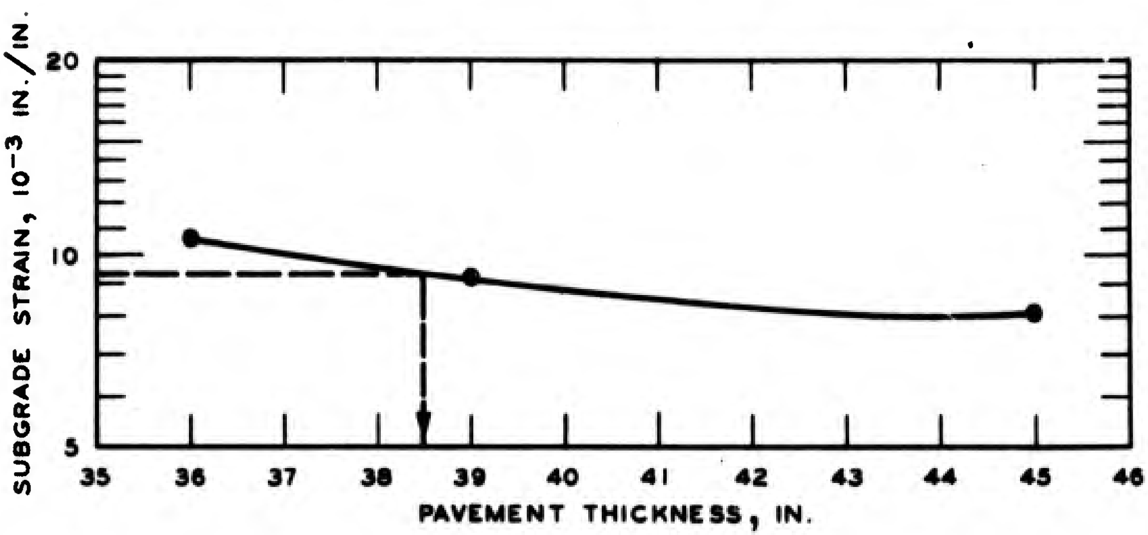


Figure K11. Relationship between subgrade strain and pavement thickness concrete strain, the number of strain repetitions is equal to the number of coverages. (The design traffic level of 200,000 total departures is equal to 108,108 coverages.) Now, if it is assumed that all of this traffic is applied during the month of January, then the allowable bituminous concrete strain determined from Figure K7 is 1.6×10^{-4} in./in.

15. For the estimated section shown in Figure K6, the bituminous concrete strain computed using the CHEVIT computer program and a design subgrade modulus of 9000 psi is 2.7×10^{-4} in./in. Since the computed strain is greater than the allowable strain, the cumulative damage must be determined.

16. To simplify the cumulative damage process, damage is accumulated for not only the estimated section shown in Figure K6 but also for two other 39-in. sections for which the bituminous concrete surface course thicknesses are 7 and 9 in. These additional sections are shown in Figures K12 and K13, respectively. (It should be noted that the increased thicknesses of the surface courses for these two additional sections are allowed for through reductions in their subbase course thicknesses.)

17. For each of the three sections, a relationship between horizontal tensile strain in the bituminous concrete and bituminous concrete modulus is developed using the CHEVIT computer program. The bituminous concrete moduli for each month from Table K1 and the section thicknesses are input to the program to compute the strain values. The curves representing the strain-modulus relationships for the three sections are plotted in Figure K14. From these curves, the horizontal tensile strain values for each month are determined, and these values and the bituminous concrete moduli are used to enter Figure K7 to determine the number of allowable strain repetitions.

18. Input and output data for the cumulative damage computations are presented in Table K3. The cumulative damage values for the sections with 5-, 7-, and 9-in. surface course thicknesses are 3.77, 1.83, and 0.84, respectively. These values are plotted versus surface course thickness in Figure K15. From this relationship, it can be estimated that the cumulative damage value equals one for a conventional flexible pavement with a bituminous concrete surface course thickness of 8.6 in. For design

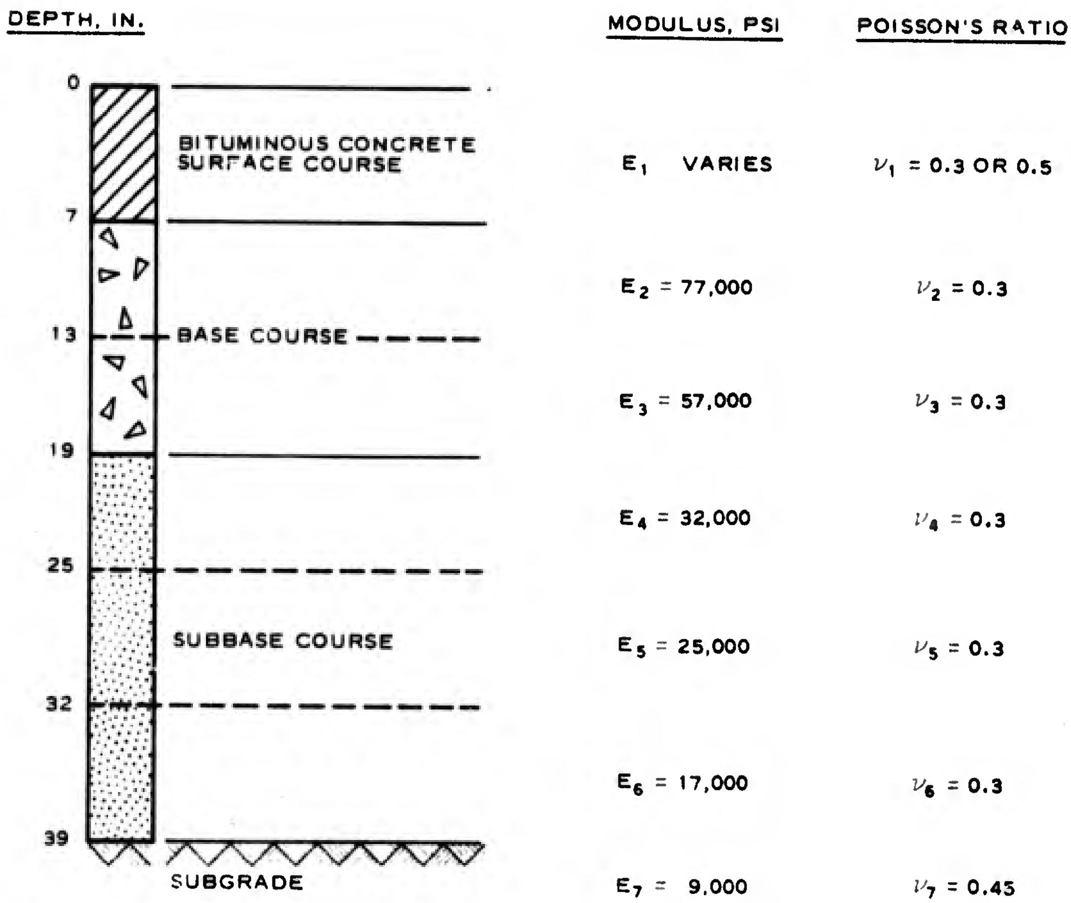


Figure K12. Estimated 39-in. conventional flexible pavement section with 7-in. bituminous concrete surface course

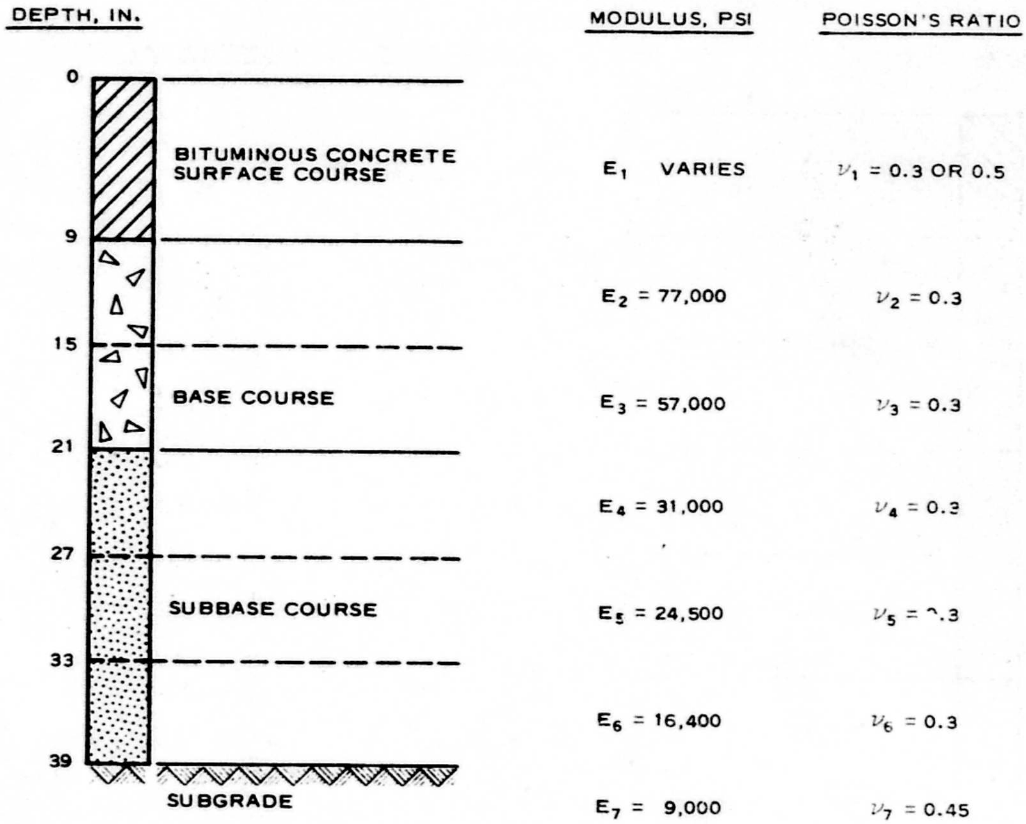


Figure K13. Estimated 39-in. conventional flexible pavement section with 9-in. bituminous concrete surface course

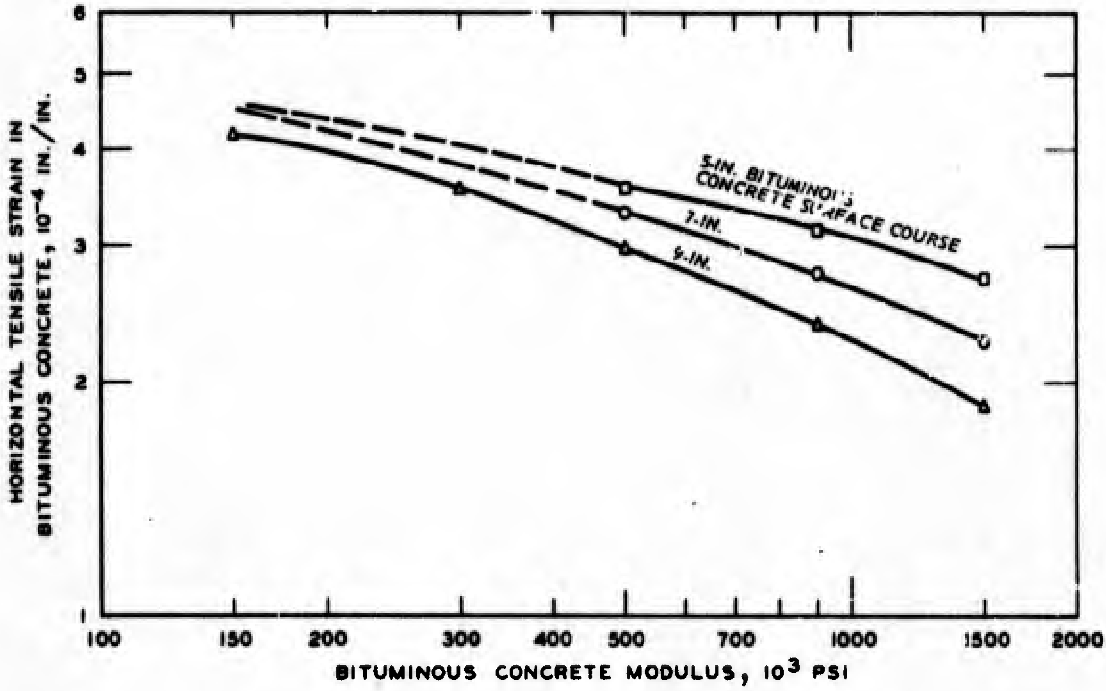


Figure K14. Relationship between computed horizontal tensile strain and bituminous concrete modulus for three estimated conventional flexible pavement sections

Table K3

Computations of Cumulative Damage for Three Conventional Flexible Pavement Sections

Traffic Period*	Bituminous Concrete Modulus 10^3 psi	Computed Horizontal Tensile Strain 10^{-4} in./in.	No. of Applied Strain Repetitions**	No. of Allowable Strain Repetitions	Damage Increment†	Cumulative Damage
			5-in. Surface Course			
1	1500	2.73	9009	11,463	0.78595	0.78595
2	1270	2.85		13,911	0.64760	1.43355
3	920	3.15		19,144	0.47059	1.90413
4	570	3.50		42,562	0.21167	2.11580
5	360	3.90		90,814	0.09920	2.21500
6	220	4.30		215,393	0.04183	2.25683
7	180	4.45		309,878	0.02907	2.28590
8	180	4.45		309,878	0.02907	2.31498
9	260	4.15		163,546	0.05509	2.37006
10	540	3.52		48,266	0.18666	2.55672
11	1000	3.06		17,795	0.50625	3.06297
12	1400	2.77		12,693	0.70976	3.77273
			7-in. Surface Course			
1	1500	2.26	9009	29,482	0.30558	0.30558
2	1270	2.42		31,515	0.28586	0.59144
3	920	2.74		38,445	0.23434	0.82578
4	570	3.20		66,621	0.13523	0.96100
5	360	3.60		135,508	0.06648	1.02749
6	220	4.12		266,739	0.03377	1.06126
7	180	4.31		363,584	0.02478	1.08604
8	180	4.31		363,584	0.02478	1.11082
9	260	3.95		209,361	0.04303	1.15385
10	540	3.25		71,934	0.12524	1.27909
11	1000	2.65		36,533	0.24660	1.52569
12	1400	2.34		29,505	0.30534	1.83103
			9-in. Surface Course			
1	1500	1.86	9009	78,078	0.11538	0.11538
2	1270	2.02		77,775	0.11583	0.23122
3	920	2.35		82,843	0.10875	0.33997
4	570	2.[illegible]		142,134	0.06338	0.40335
5	360	3.33		200,104	0.04502	0.44837
6	220	3.90		350,954	0.02567	0.47404
7	180	4.06		490,186	0.01838	0.49242
8	180	4.06		490,186	0.01838	0.51080
9	260	3.72		282,596	0.03188	0.54268
10	540	2.80		151,551	0.05945	0.60212
11	1000	2.27		79,211	0.11373	0.71586
12	1400	1.94		75,329	0.11960	0.83545

* Traffic period 1 consists of the 20 January's in the 20-yr design life, 2 consists of the 20 February's, etc.

** The number of applied strain repetitions is 200,000 (the number of total departures) divided by 1.85 (the factor for converting departures to coverages) divided by 12 (the number of traffic periods).

† The damage increment is the number of applied repetitions divided by the number of allowable repetitions.

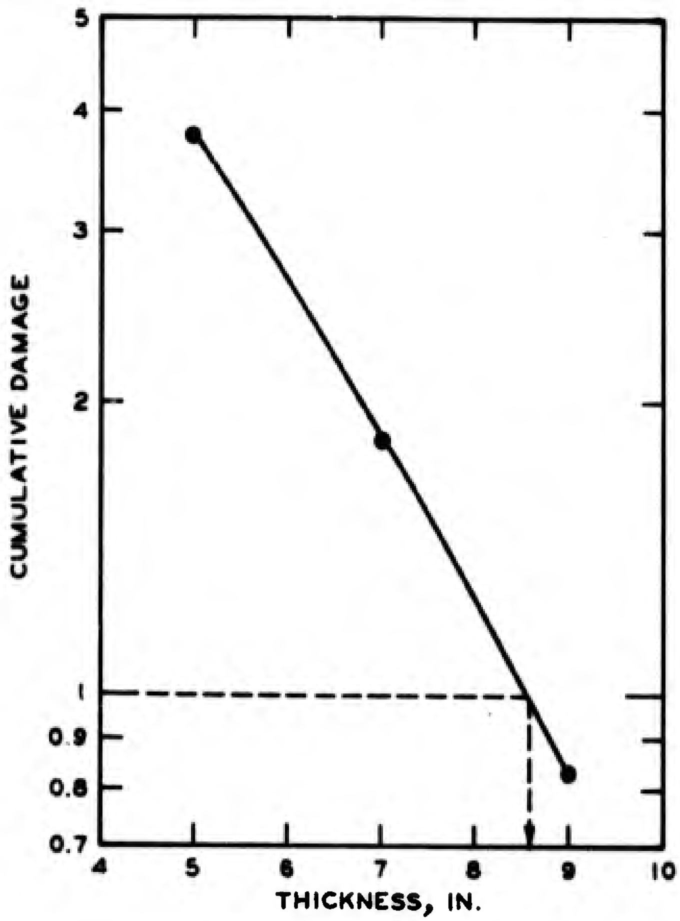


Figure K15. Relationship between cumulative damage and bituminous concrete surface course thickness (conventional flexible pavement) purposes, this value would be rounded to 9 in.

19. A histogram of the fatigue damage incurred by the section with the 9-in. bituminous concrete surface course is shown in Figure K16. It

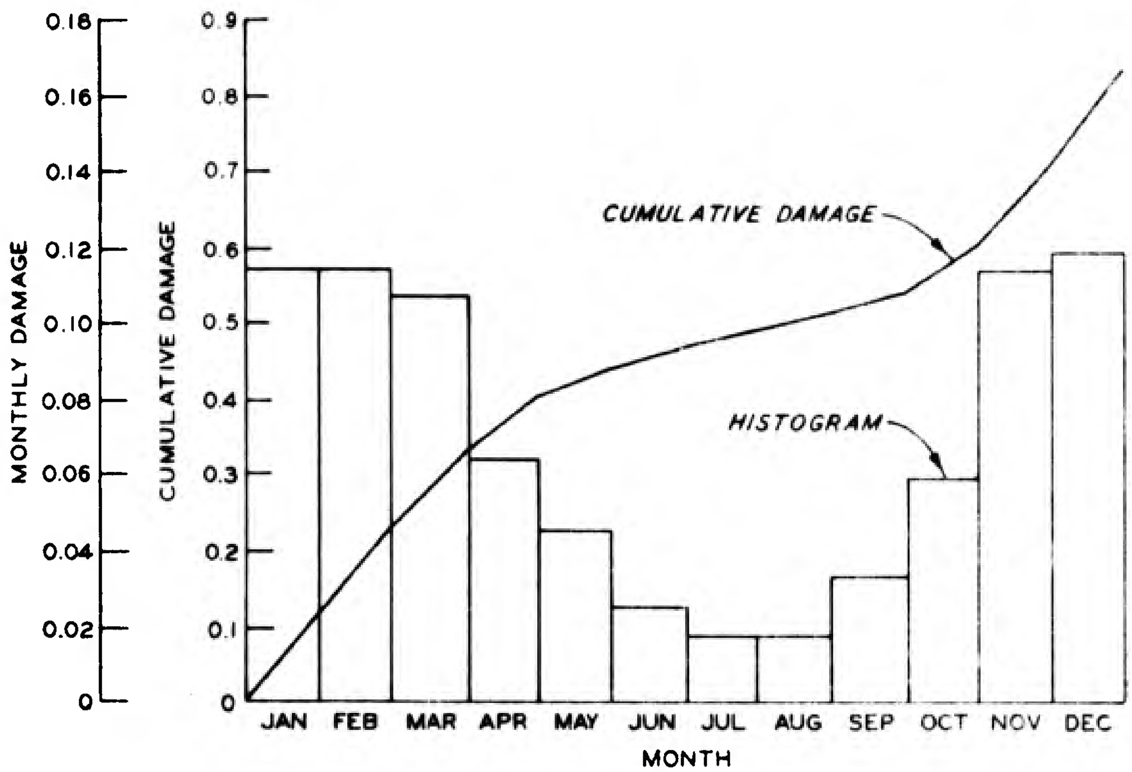


Figure K16. Histogram of fatigue damage incurred by estimated section with 9-in. bituminous concrete surface course

should be noted that a significant portion of the damage occurs during the colder months.

20. The performance model indicates that the total thickness of conventional flexible pavement required to limit vertical compressive strain at the top of the subgrade is 39 in. and that the bituminous concrete thickness required to limit horizontal tensile strain in this layer is 9 in. The final design is therefore identical with the section shown in Figure K13. However, it may be possible to reduce the total thickness due to the increased surface course thickness. To determine if a reduction is possible, the subgrade strain is recomputed to establish a new minimum thickness.

the acceptability of the new design must also be checked by means of the bituminous concrete strain criteria.

PROBLEM 2: ABC PAVEMENT

21. To design an ABC pavement, the performance model illustrated in Figure K17 is used. Input parameters required for this model are:

- a. Temperature data.
- b. Subgrade modulus.
- c. Traffic parameters.
- d. Limiting strain criteria.
- e. Estimated initial thickness.

TRAFFIC PARAMETERS

22. The traffic parameters used in this design are the same as those used for the conventional flexible pavement.

ESTIMATED INITIAL THICKNESS

23. To estimate the initial thickness of an ABC pavement, the required thickness for the conventional flexible pavement is divided by the appropriate equivalency factor (1.70) from Table 2 of the main text of this report. The estimated initial thickness is therefore 39 in. ÷ 1.70 = 22 in.

BITUMINOUS CONCRETE MODULUS

24. The bituminous concrete modulus for each month is determined in the same manner as that for the conventional flexible pavement. Tables K4 and K5 present the temperature data and the modulus values for each month for design based on bituminous concrete strain and subgrade strain, respectively.

SUBGRADE MODULUS

25. The subgrade modulus used in the ABC pavement design is 9000 psi.

LIMITING STRAIN CRITERIA

26. The limiting strain criteria are shown in Figures K7 and K8.

ANALYSIS OF DESIGN SECTION

27. The cumulative damage process is used to determine if the 22-in. estimated section for ABC pavement is an acceptable design with respect to

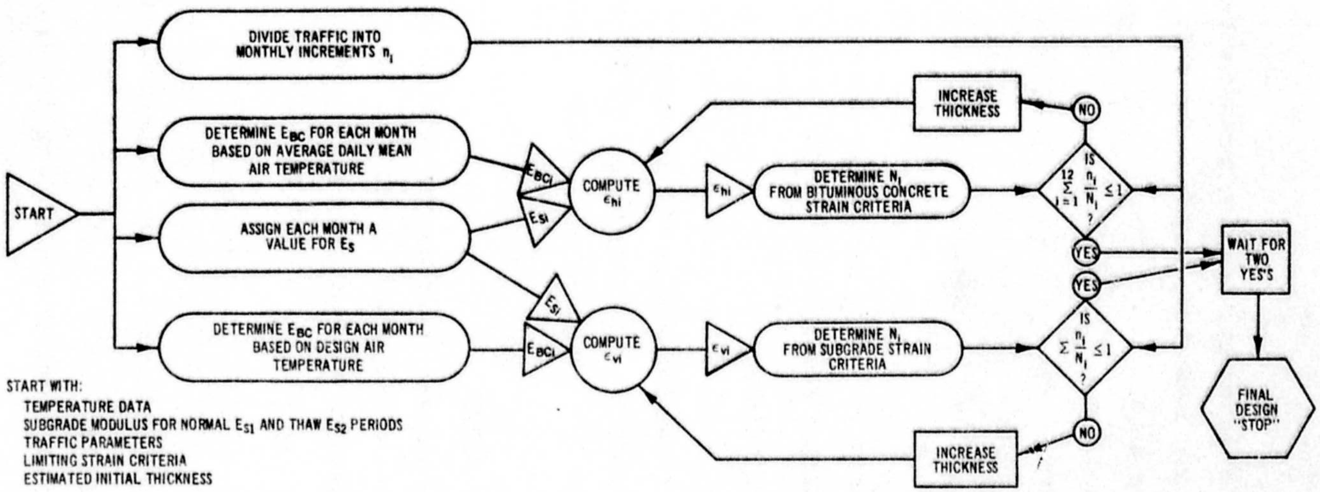


Figure K17. Flow diagram for ABC pavement

Table K4

Bituminous Concrete Moduli for Each Month
For ABC Pavement Design Based on Bituminous Concrete Strain

Month	Average Daily Mean Air Temperature °F	Design Pavement Temperature °F	Dynamic Modulus $\|E^*\|$ 10^3 psi
Jan	47.5	54	1600
Feb	50.7	57	1400
Mar	58.0	64	1060
Apr	66.1	72	700
May	73.3	80	460
Jun	80.5	88	280
Jul	83.1	91	230
Aug	82.7	91	230
Sep	77.3	85	340
Oct	67.2	73	670
Nov	56.2	61	1200
Dec	49.3	56	1500

Table K5

Bituminous Concrete Moduli for Each Month
For ABC Pavement Design Based on Subgrade Strain

Month	Average Daily Mean Air Temperature °F	Average Daily Maximum Air Temperature °F	Design Air Temperature °F	Design Pavement Temperature °F	Dynamic Modulus $\|E^*\|$ 10^3 psi
Jan	47.5	56.4	52	57	1400
Feb	50.7	60.1	55	62	1150
Mar	58.0	68.0	63	70	790
Apr	66.1	76.0	71	77	540
May	73.3	83.2	78	86	320
Jun	80.5	90.4	85	95	180
Jul	83.1	92.9	88	97	160
Aug	82.7	92.8	88	97	160
Sep	77.3	87.4	82	91	230
Oct	67.2	78.1	73	82	400
Nov	56.2	66.4	61	69	830
Dec	49.3	58.3	54	61	1200

limiting strain at the bottom of the bituminous concrete and at the top of the subgrade. As the first step in the process, the design traffic is divided into monthly periods for consideration of subgrade strain and into traffic periods for consideration of bituminous concrete strain. For an ABC pavement, 1 departure constitutes 1 strain repetition, whether the design is based on subgrade strain or bituminous concrete strain. Thus, the number of applied strain repetitions in each period for consideration of subgrade strain is the number of annual departures (10,000) divided by 12, or 833; and the number of applied strain repetitions in each period for consideration of bituminous concrete strain is the total number of departures (200,000) divided by the number of traffic periods (12), or 16,666.

28. The other input parameter required in the cumulative damage relations is the number of allowable strain repetitions for each of the periods described above. To determine these values, the bituminous concrete and subgrade strains must first be computed. As was illustrated in the computations for Problem 1, the cumulative damage process can be simplified by accumulating damage for several pavement thicknesses. This approach is also followed for the ABC pavement; therefore, the bituminous concrete and subgrade strains are computed for selected ABC pavement thicknesses and a range of bituminous concrete moduli. Curves representing the strain-modulus relationships for the selected thicknesses are plotted in Figures K18 and K19. From these curves, the bituminous concrete and subgrade strain values for each period are determined, and these values are used to enter Figures K7 and K8, respectively, to determine the number of allowable strain repetitions for each period.

29. Input and output data for the cumulative damage computations for fatigue damage in the bituminous concrete are presented in Table K6 for 18-, 22-, and 26-in. ABC pavement sections. The cumulative damage values for these sections are 0.56, 0.14, and 0.05, respectively. These values are plotted versus ABC thickness in Figure K20. From this relationship, it can be estimated that the cumulative damage value equals one for an ABC thickness of 17.2 in. For design purposes, this value would be rounded to 18 in., which is the required thickness to limit horizontal

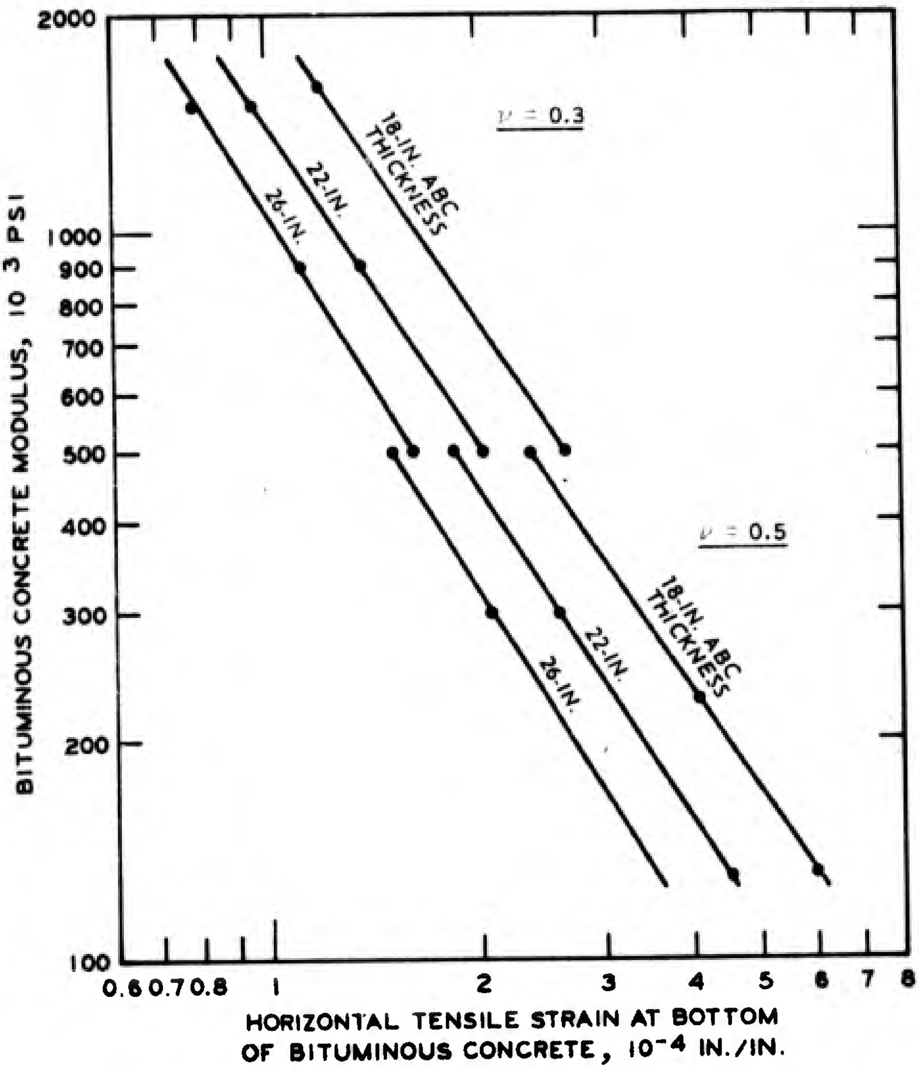


Figure K18. Relationship between computed horizontal tensile strain and bituminous concrete modulus for three ABC pavement sections

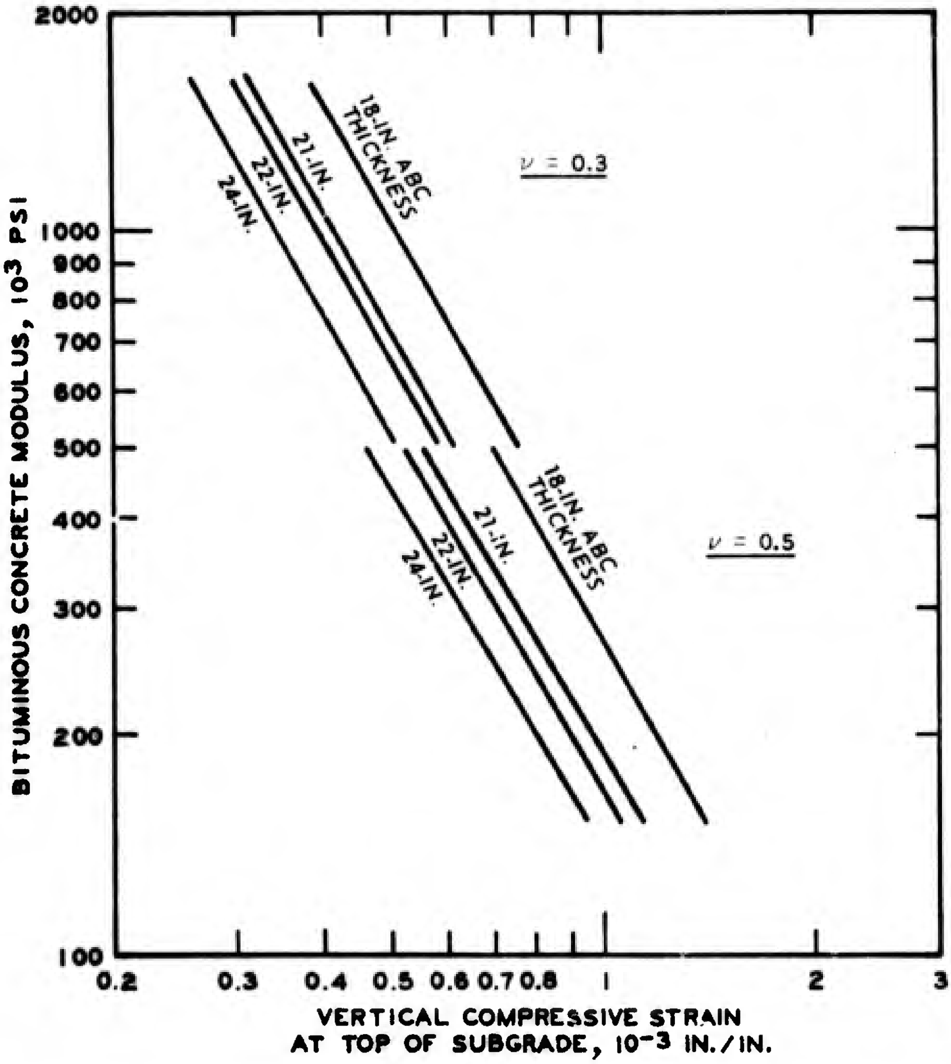


Figure K19. Relationship between computed vertical compressive strain and bituminous concrete modulus for four ABC pavement sections

Table K6

Computations of Cumulative Damage of the Bituminous Concrete for Three ABC Pavement Sections

Traffic Period	Bituminous Concrete Modulus 10^3 psi	Computed Horizontal Tensile Strain 10^{-4} in./in.	No. of Applied Strain Repetitions	No. of Allowable Strain Repetitions	Damage Increment	Cumulative Damage
			18-in. ABC Pavement			
1	1600	1.19	16,666	616,171	0.02705	0.02705
2	1400	1.30	↓	557,508	0.02989	0.05694
3	1060	1.57		430,789	0.03869	0.09563
4	700	2.10		303,983	0.05483	0.15045
5	460	2.52		404,696	0.04118	0.19164
6	280	3.60		271,805	0.06132	0.25295
7	230	4.10		242,430	0.06875	0.32170
8	230	4.10		242,430	0.06875	0.39044
9	340	3.12		325,013	0.05128	0.44172
10	670	2.15		306,293	0.05441	0.49613
11	1200	1.45		469,095	0.03553	0.53166
12	1500	1.25		569,568	0.02926	0.56092
			22-in. ABC Pavement			
1	1600	0.81	16,666	4,217,056	0.00395	0.00395
2	1400	1.00	↓	2,069,987	0.00805	0.01200
3	1060	1.20		1,651,418	0.01009	0.02210
4	700	1.61		1,147,669	0.01452	0.03662
5	460	1.94		1,496,656	0.01114	0.04775
6	280	2.70		1,145,385	0.01455	0.06230
7	230	3.08		1,013,333	0.01645	0.07875
8	230	3.08		1,013,333	0.01645	0.09520
9	340	2.37		1,285,085	0.01297	0.10817
10	670	1.65		1,150,563	0.01449	0.12265
11	1200	1.10		1,866,973	0.00893	0.13158
12	1500	0.95		2,246,338	0.00742	0.13900
			26-in. ABC Pavement			
1	1600	0.76	16,666	5,799,176	0.00287	0.00287
2	1400	0.83	↓	5,255,036	0.00317	0.00605
3	1060	1.00		4,109,259	0.00406	0.01010
4	700	1.30		3,343,701	0.00498	0.01509
5	460	1.57		4,311,544	0.00387	0.01895
6	280	2.26		2,787,587	0.00598	0.02493
7	230	2.46		3,117,671	0.00535	0.03028
8	230	2.46		3,117,671	0.00535	0.03562
9	340	1.80		5,085,233	0.00328	0.03890
10	670	1.35		3,138,067	0.00531	0.04421
11	1200	0.92		4,562,078	0.00365	0.04786
12	1500	0.80		5,304,486	0.00314	0.05100

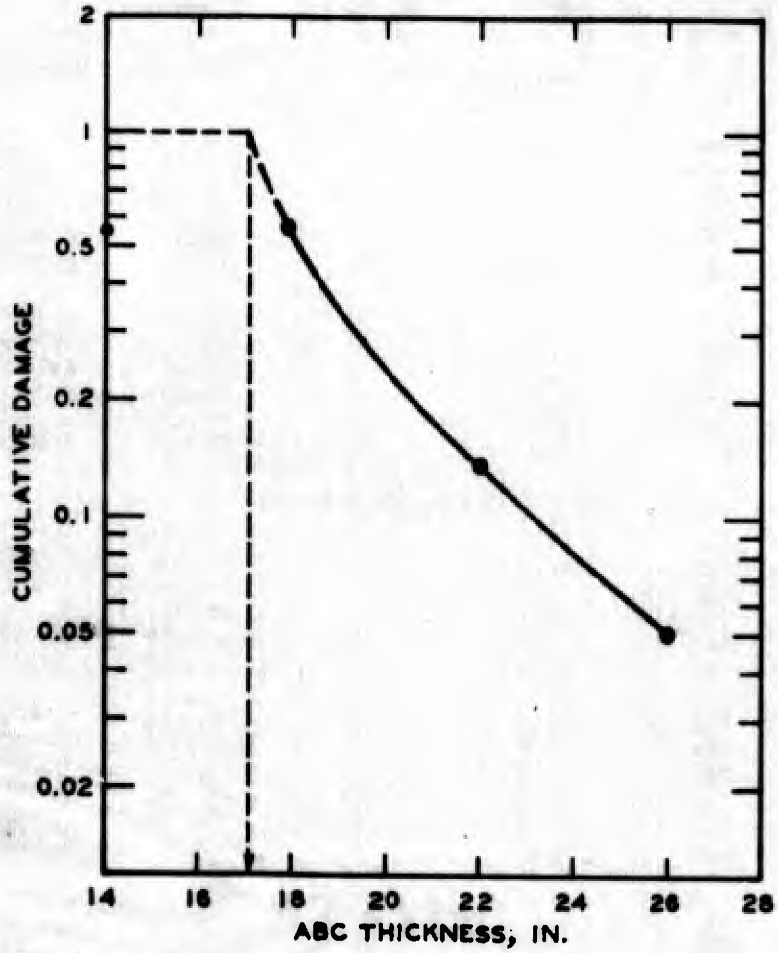


Figure K20. Relationship between cumulative damage and ABC thickness (for design based on bituminous concrete strain tensile strain at the bottom of the bituminous concrete.

30. A histogram of the fatigue damage incurred by the 18-in. ABC pavement is shown in Figure K21. It should be noted that significant

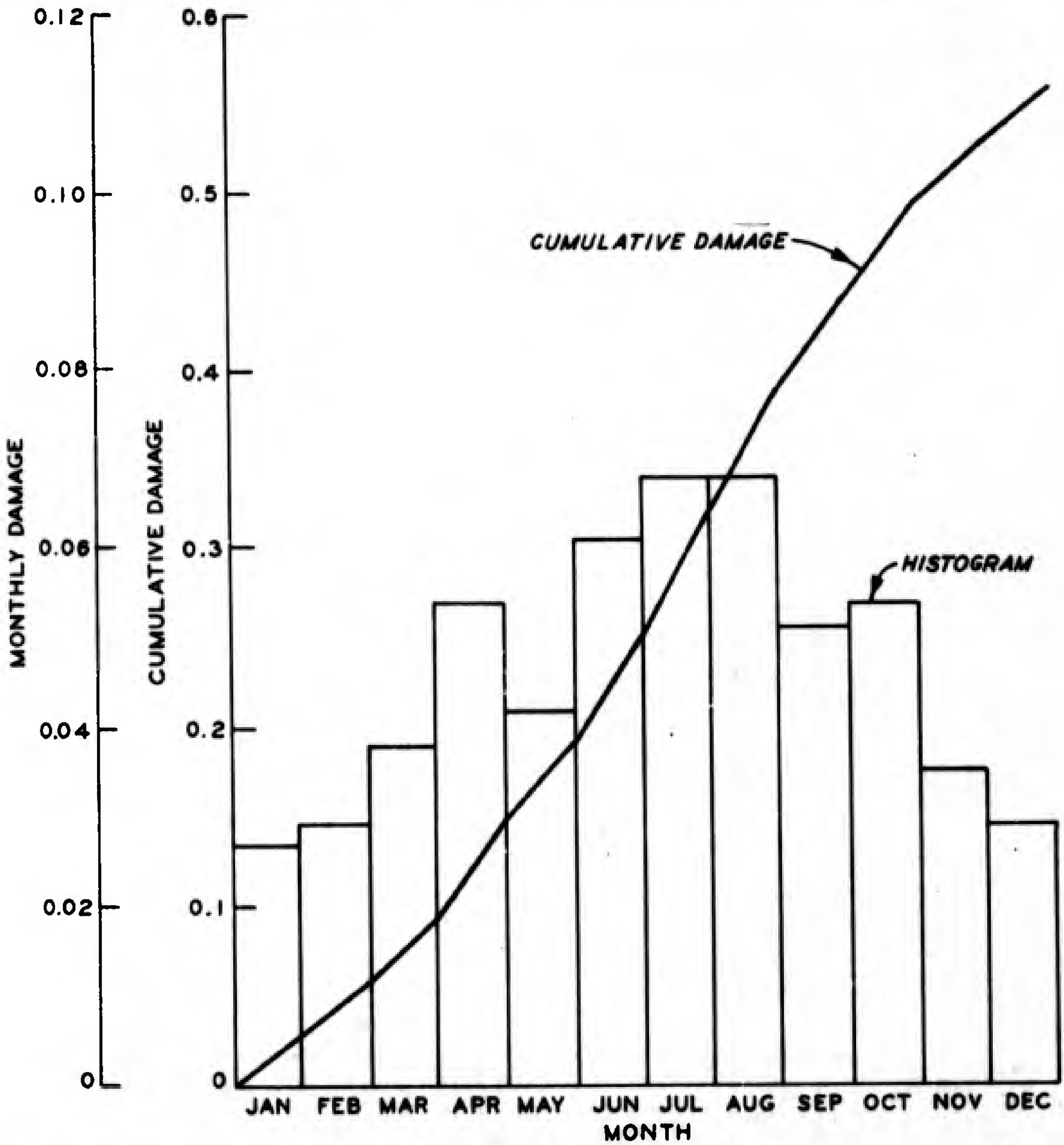


Figure K21. Histogram of fatigue damage incurred by 18-in. ABC pavement

increments of damage occur during each month and that the maximum fatigue damage occurs during the warmer months. This behavior is in contrast with that for the conventional flexible pavement, in which the maximum fatigue damage occurs during the colder months.

31. Input and output data for computations of cumulative damage to the subgrade are presented in Table K7 for 18-, 21-, 22-, and 24-in. ABC pavements. The data indicate that the damage caused by a subgrade strain less than 0.78×10^{-3} in./in. is negligible; therefore, damage is not computed for months during which the subgrade strain is less than this value. The cumulative damage values for the four sections are 10.18, 1.25, 0.50, and 0.12, respectively. These values are plotted versus ABC thickness in Figure K22. From this relationship, it can be estimated that the

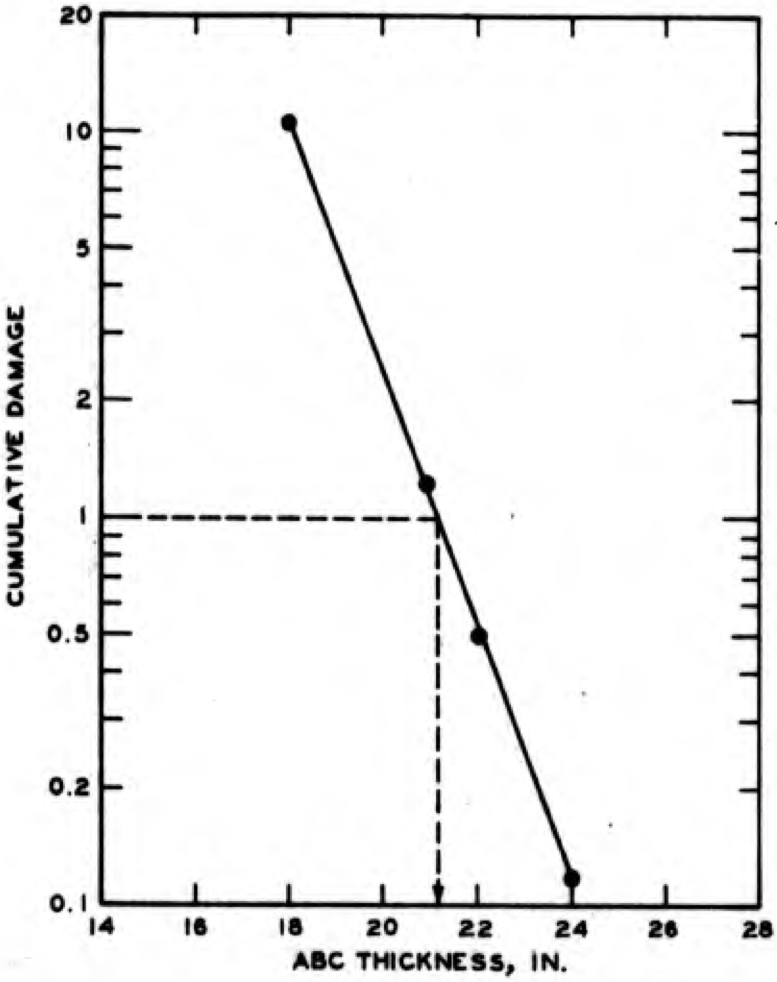


Figure K22. Relationship between cumulative damage and ABC thickness (for design based on subgrade strain)

cumulative damage value equals one for an ABC thickness of slightly more than 21 in., which is the required thickness to limit vertical compressive

Table K7

Computations of Cumulative Damage of the Subgrade for Four ABC Pavement Sections

Monthly Period	Bituminous Concrete Modulus 10^3 psi	Computed Vertical Compressive Strain 10^{-3} in./in.	No. of Applied Strain Repetitions	No. of Allowable Strain Repetitions	Damage Increment	Cumulative Damage
			18-in. ABC Pavement			
May	320	0.90	833	20,000	0.04	0.04
Jun	180	1.20	↓	630	1.32	1.36
Jul	160	1.35	↓	200	4.16	5.52
Aug	160	1.35	↓	200	4.16	9.68
Sep	230	1.10	↓	1,700	0.49	10.17
Oct	400	0.79	↓	90,000	0.01	10.18
			21-in. ABC Pavement			
Jun	180	1.02	833	4,000	0.21	0.21
Jul	160	1.10	833	1,650	0.50	0.71
Aug	160	1.10	833	1,650	0.50	1.21
Sep	230	0.89	833	22,000	0.04	1.25
			22-in. ABC Pavement			
Jun	180	0.94	833	11,500	0.07	0.07
Jul	160	1.02	833	4,000	0.21	0.28
Aug	160	1.02	833	4,000	0.21	0.49
Sep	230	0.82	833	60,000	0.01	0.50
			24-in. ABC Pavement			
Jun	180	0.86	833	34,000	0.02	0.02
Jul	160	0.92	833	15,000	0.05	0.07
Aug	160	0.92	833	15,000	0.05	0.12

Note: Damage was accumulated only for those months during which the computed subgrade strain was greater than or equal to 0.78×10^{-3} in./in.

strain at the top of the subgrade.

32. Thus, for the ABC pavement alternative, the subgrade strain criteria control the design, and the required thickness is 22 in.

PROBLEM 3: CONVENTIONAL FLEXIBLE PAVEMENT WITH CHEMICALLY STABILIZED SUBGRADE

33. The conventional flexible pavement to be designed will consist of a bituminous concrete surface course, unbound base and subbase courses, and a chemically stabilized subgrade layer over the natural soil. A resilient modulus test conducted on laboratory prepared specimens of the stabilized soil using the procedures for subgrade soils outlined in Appendix C indicates that the modulus value for this material is 50,000 psi. In the selection of the design subgrade modulus, however, consideration must be given to the difference between the value determined from laboratory tests and the actual field modulus value. This approach is taken since, in general, a laboratory prepared specimen is far more uniform in mix distribution, density, etc., than is the material in a large field construction project. To account for this difference, the design modulus of the stabilized subgrade is taken to be 50 percent of the value obtained in the laboratory, or 25,000 psi.

34. It is also assumed that the modulus of the lower most 6 in. of stabilized soil is the same as that of the subgrade, i.e., 9000 psi. This assumption is made since, during field construction of the initial layer of stabilized soil, it is not possible to obtain as high a degree of density as is achieved during compaction of the next overlying layer. In essence, this implies that the first layer of stabilized soil provides a working platform for subsequent construction.

35. In this design, the thicknesses of the bituminous concrete surface course and the base course are the same as those selected for the design of the conventional flexible pavement in Problem 1, i.e., 9 and 12 in., respectively. Also the design thickness of the subbase course is determined so that the modulus value of the upper part of this layer is approximately the same as that of the subbase course in Problem 1, i.e., 31,000 psi. This approach is taken so that the base course and surface course will have the same thickness values and the bituminous concrete

will meet fatigue criteria as determined in the conventional flexible pavement design.

36. Next, it is necessary to determine whether the thicknesses of the materials above the stabilized and natural subgrade are sufficient to limit strains in these layers to acceptable values. A thickness of 12 in. of stabilized material is first assumed. This thickness does not include the 6 in. allowed for the working platform. The resulting estimated section is shown in Figure K23. The subgrade strain criteria indicate an

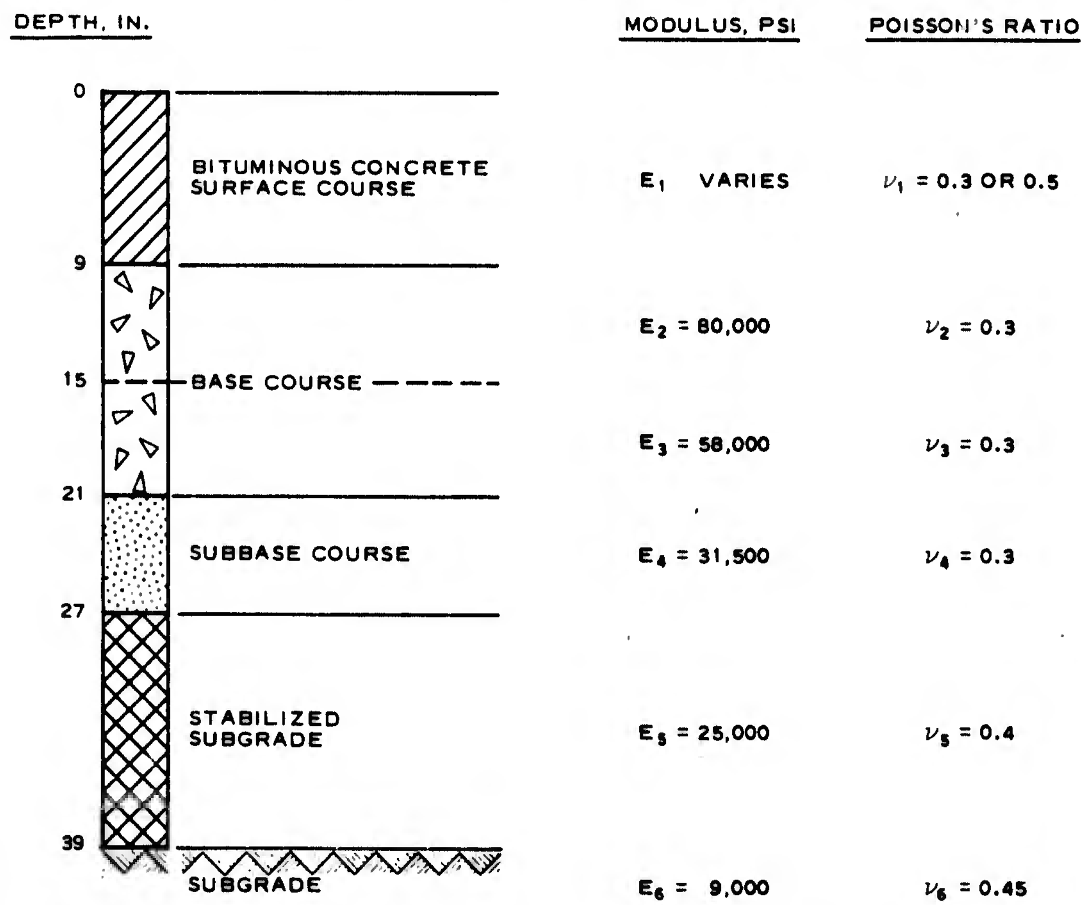


Figure K23. Estimated section for conventional flexible pavement with stabilized subgrade

allowable value of 1.10×10^{-3} in./in. for the stabilized subgrade and an allowable value of 0.94×10^{-3} in./in. for the natural subgrade. The computed strain values at the surface of the stabilized and natural subgrade layers are 0.75×10^{-3} and 0.90×10^{-3} in./in., respectively. Since both of the computed strain values are less than the allowable values, the initial design is satisfactory. It would appear that since the computed strain value at the top of the stabilized subgrade is much lower than the allowable value, the design thickness of the base or subbase course could be reduced. However, no reduction in the design thickness of these layers can be made due to the thickness required by the fatigue criteria for the bituminous concrete surface course. Therefore, the final design is as shown in Figure K24.

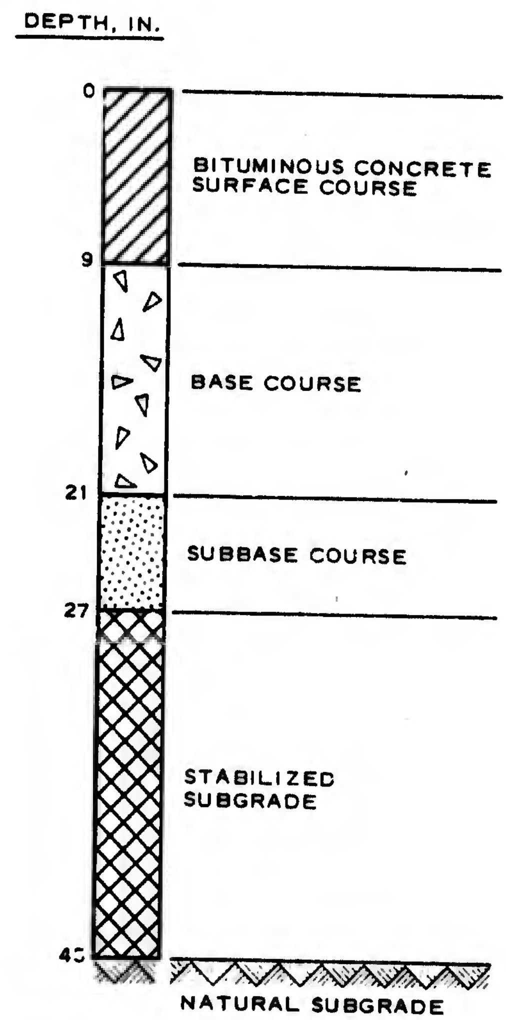


Figure K24. Design section for conventional flexible pavement with stabilized subgrade

PROBLEM 4: CHEMICALLY STABILIZED PAVEMENT WITH STABILIZED BASE AND SUBBASE COURSES

37. To design a chemically stabilized pavement in which the base and subbase courses are stabilized, the performance model illustrated in Figure K25 is used. In this design, it is assumed that, to meet specified strength criteria, the base and subbase courses are stabilized with 6 and 4 percent portland cement, respectively. The initial estimated design section is shown in Figure K26.

38. First, it is necessary to determine whether the base course will crack under loading. For this determination, the lowest estimated modulus value of the bituminous concrete, the flexural modulus value of the base course, and the cracked section modulus value of the subbase course are used. The design section applicable for this determination is shown in Figure K27. Calculation of the horizontal tensile strain at the bottom of the base course indicates a value of 1.7×10^{-4} in./in., which, when multiplied by 1.5 to account for shrinkage cracking, yields a computed strain value of 2.55×10^{-4} in./in. From Figure 29 in the main text, the allowable horizontal strain is 0.7×10^{-4} in./in. Since the allowable strain value is less than the computed value, a cracked section base course modulus must be used for subsequent computations.

39. Using the cracked section modulus for the base course, the next step is to determine whether the subbase course will crack. For these computations, the flexural modulus of the subbase course is used. The estimated section for this determination is the same as that shown in Figure K27, except that the modulus values of the base and subbase courses are 150,000 and 500,000 psi, respectively. Computation of the horizontal tensile strain at the bottom of the subbase course indicates a value of 1.6×10^{-4} in./in., which, when multiplied by 1.5, yields a computed strain value of 2.4×10^{-4} in./in. The allowable strain is again 0.7×10^{-4} in./in.; therefore, for consideration of subgrade strain, the design section must be evaluated assuming that both the base and subbase courses are cracked. The estimated section for this determination is shown in Figure K28.

40. Using the cracked section modulus values, the vertical

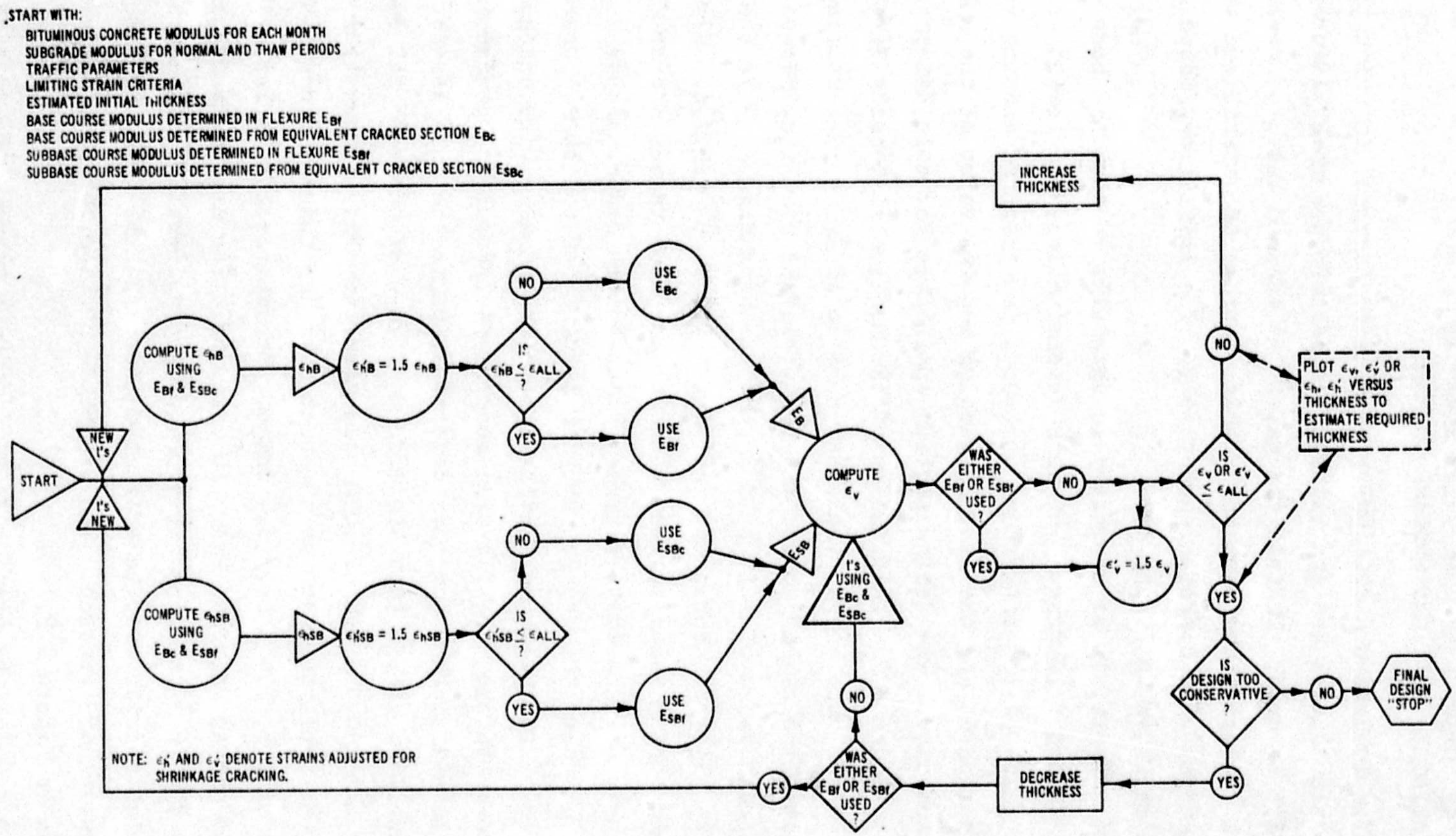


Figure K25. Flow diagram for chemically stabilized pavement having stabilized base and subbase course

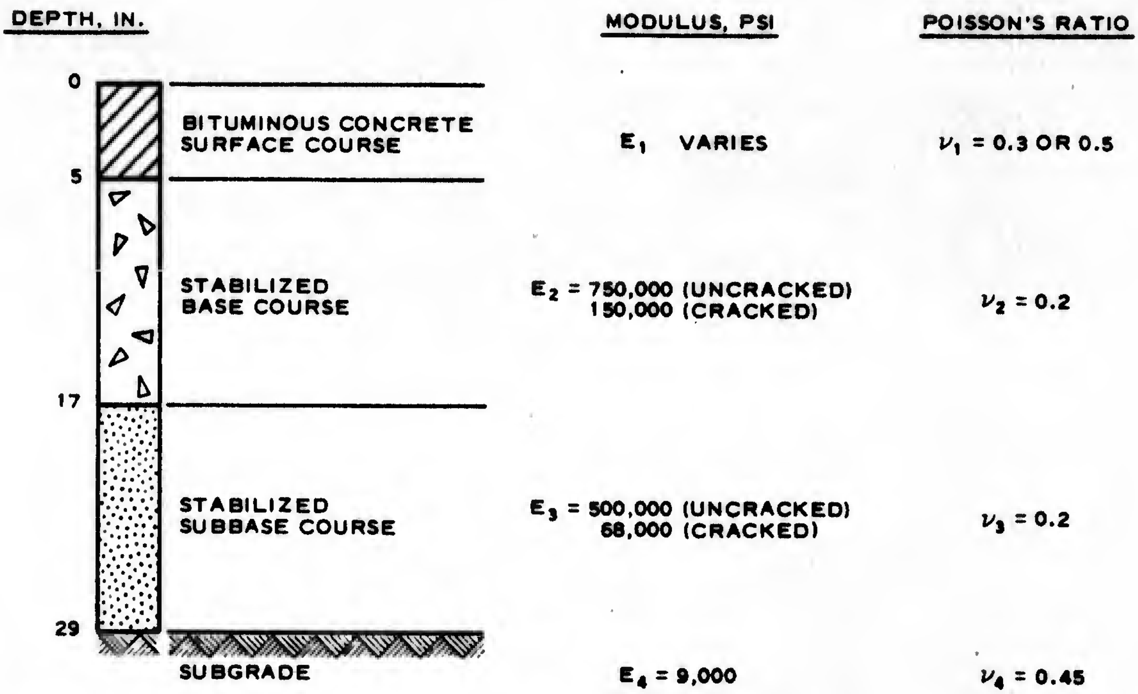


Figure K26. Estimated section for chemically stabilized pavement with stabilized base and subbase courses

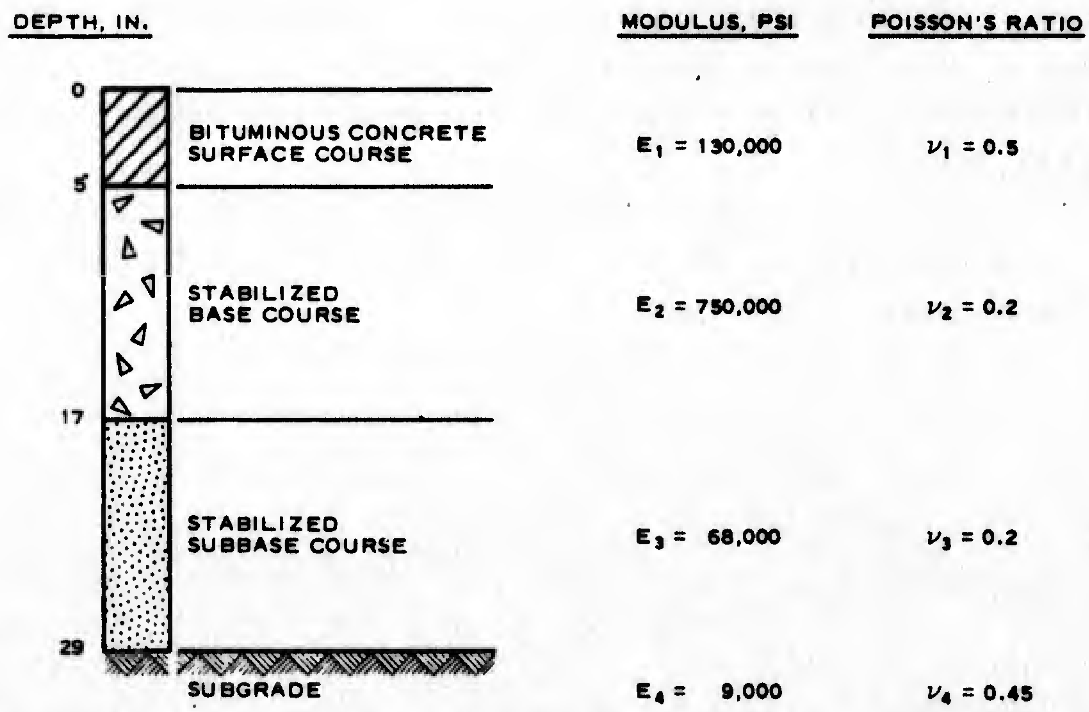


Figure K27. Estimated section for checking cracking of stabilized base course

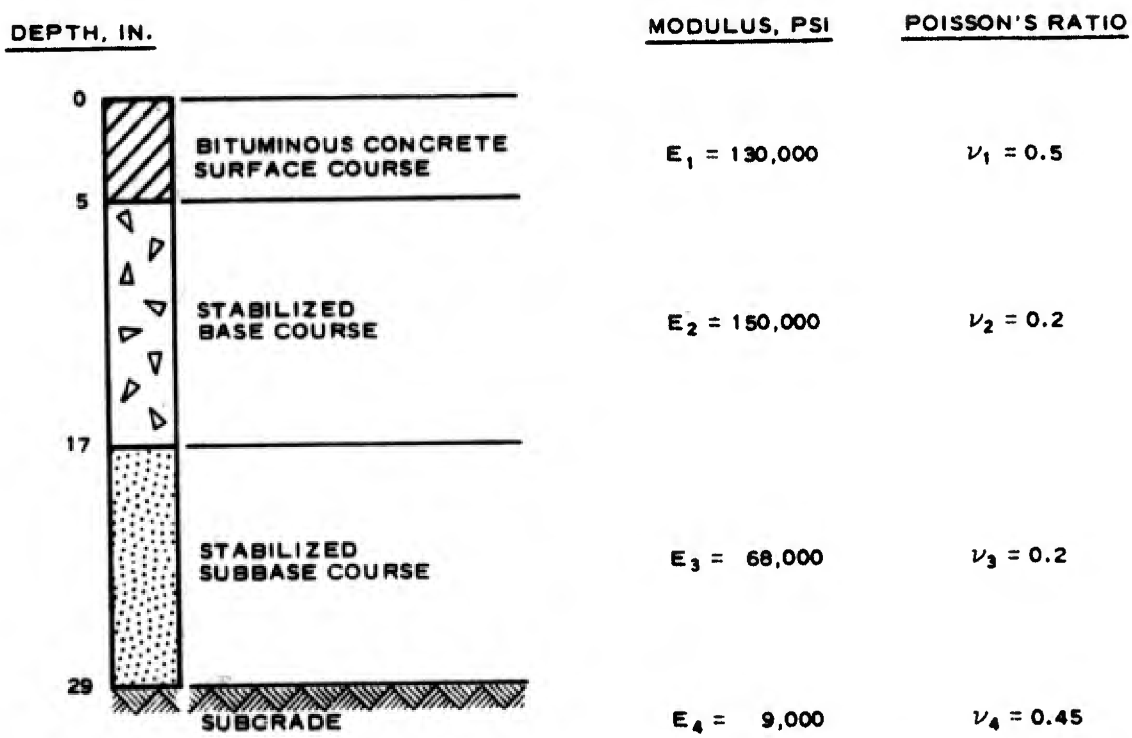


Figure K28. Estimated section for considering subgrade strain

compressive strain at the top of the subgrade is computed as 9.4×10^{-4} in./in., which is equal to the allowable strain value. Therefore, the design section shown in Figure K28 is satisfactory with respect to subgrade strain.

41. The initial design section shown in Figure K26 would therefore be used in design since the thicknesses of this section are sufficient to satisfy all of the criteria.

REFERENCES

1. Izatt, J. O., Lettier, J. A., and Taylor, C. A., "The Shell Group Methods for Thickness Design of Asphalt Pavements," Paper presented at the Annual Meeting of the National Asphalt Paving Association, Jan 1967, San Juan, Puerto Rico.

2. Michelow, J., "Analysis of Stresses and Displacements in an n-Layered Elastic System Under a Load Uniformly Distributed on a Circular Area," Sep 1963, California Research Corporation, Richmond, Calif.

3. Harrison, W. J., Wardle, L. J., and Gerrard, C. M., "Computer Programmes for Circle and Strip Loads on Layered Anisotropic Media," Geomechanics Computing Programme N.1, 1972, Division of Applied Geomechanics, Commonwealth Scientific and Industrial Research Organization, Melbourne, Australia.

4. Witczak, M. W., "Design of Full-Depth Asphalt Airfield Pavements," Research Report 72-2, Apr 1972, The Asphalt Institute, College Park, Md.

5. Burns, C. D., Ledbetter, R. H., and Grau, R. W., "Study of Behavior of Bituminous-Stabilized Pavement Layers," Miscellaneous Paper S-73-4, Mar 1973, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

6. Pretorius, P. C., "Design Considerations for Pavements Containing Soil-Cement Bases," Ph.D. Dissertation, 1969, University of California, Berkeley, Calif.

7. Brabston, W. N., Barker, W. R., and Harvey, G. G., "Development of a Structural Design Procedure for All-Bituminous Concrete Pavements for Military Roads," Technical Report S-75-10, Jul 1975, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

8. National Oceanic and Atmospheric Administration, Environmental Data Service, "Local Climatological Data Annual Summary with Comparative Data, Jackson, Mississippi," 1973, National Climatic Center, Asheville, N. C.

9. Federal Aviation Administration, "Airport Pavement Design and Evaluation," Advisory Circular AC 150/5320-6B, May 1974, Washington, D. C.

10. Headquarters, Department of the Army, "Soils and Geology: Pavement Design for Frost Conditions," Technical Manual TM 5-818-2, Jul 1965, Washington, D. C.

11. Monismith, C. L. and Finn, F. N., "Moderators' Summary Report of Papers Presented for Discussion at Session III-Design Theory," Third International Conference on the Structural Design of Asphalt Pavements, 1972, London.

12. Fossberg, P. E., "Load-Deformation Characteristics of Three-Layer Pavements Containing Cement-Stabilized Base," Ph.D. Dissertation, 1970, University of California, Berkeley, Calif.

13. Headquarters, Department of the Army, "General Provisions for Airfield Design," Technical Manual TM 5-824-1 (in preparation), Washington, D. C.

14. Brown, D. N. and Thompson, O. O., "Lateral Distribution of Aircraft Traffic," Miscellaneous Paper S-73-56, Jul 1973, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

15. Hammitt, G. M., Barker, W. R., and Rone, C. L., "Comparative Performance of Structural Layers in Pavement Systems; Analysis of Test Section Data and Presentation of Design and Construction Procedures," Report No. FAA-RD-73-198, Vol II (in preparation), Federal Aviation Administration, Washington, D. C.

16. Federal Aviation Administration, "Standards for Specifying Construction of Airports," Advisory Circular AC 150/5370-10, Oct 1974, Washington, D. C.

17. Headquarters, Department of the Army, "Bituminous Intermediate and Surface Courses for Airfields, Heliports, and Tank Roads (Central-Plant Hot-Mix)," Guide Specification for Military Construction CE-807.22, Dec 1972, Washington, D. C.

18. Kallas, B. F. and Riley, J. C., "Mechanical Properties of Asphalt Pavement Materials," Proceedings, Second International Conference on the Structural Design of Asphalt Pavements, 1967, Ann Arbor, Mich.

19. Shook, J. F. and Kallas, B. F., "Factors Influencing Dynamic Modulus of Asphalt Concrete," Proceedings, Association of Asphalt Paving Technologists, Vol 38, 1969.

20. Papazian, H. S., "The Response of Linear Viscoelastic Materials in the Frequency Domain with Emphasis on Asphaltic Concrete," Proceedings, International Conference on the Structural Design of Asphalt Pavements, 1962, Ann Arbor, Mich.

21. Finn, F. N., "Factors Involved in the Design of Asphaltic Pavement Surface," National Highway Corporation Research Program Report No. 39, 1967, Highway Research Board, Washington, D. C.

22. Van der Poel, C., "A General System Describing the Viscoelastic Properties of Bitumens and Its Relation to Routine Test Data," Journal of Applied Chemistry, Vol 4, Part 5, May 1954.

23. Heukelom, W. and Klomp, A. J. G., "Road Design and Dynamic Loading," Proceedings, Association of Asphalt Paving Technologists, Vol 33, 1964.

24. Van Draat, W. E. F. and Sommer, P., "Ein Gerat zur Bestimmunger der dynamischen Elastizitats modulen von Asphalt," Strasse and Autobahn, Vol 35, 1966, Bonn.

25. Wang, M., "Stresses and Deflections in Cement Stabilized Soil Pavements," Ph.D. Dissertation, 1968, University of California, Berkeley, Calif.

26. American Society for Testing and Materials, 1974 Annual Book of ASTM Standards, Part 15, 1974, Philadelphia, Pa.

27. Seed, H. B. and Fead, J. W. N., "Apparatus for Repeated Load Tests on Soils," Special Technical Publication No. 254, 1959, American Society of Testing and Materials, Philadelphia, Pa.

28. Seed, H. B., Chan, C. K., and Lee, C. E., "Resilience Characteristics of Subgrade Soils and Their Relation to Fatigue Failures in Asphalt Pavements," Proceedings, International Conference on the Structural Design of Asphalt Pavements, 1962, Ann Arbor, Mich.

29. Deacon, J. A. and Monismith, C. L., "Laboratory Flexural Fatigue Testing of Asphalt Concrete with Emphasis on Compound Loading Tests," Highway Research Record No. 158, 1967, Highway Research Board, Washington, D. C.

30. Heukelom, W. and Klomp, A. J. G., "Dynamic Testing as a Means of Controlling Pavements During and After Construction," Proceedings, International Conference on the Structural Design of Asphalt Pavements, 1962, Ann Arbor, Mich.

31. Fossberg, P. E., Mitchell, J. K., and Monismith, C. L., "Cracking and Edge Loading Effects on Stresses and Deflections in a Soil-Cement Pavement," Paper presented at the 51st Annual Meeting of the Highway Research Board, 1972.

32. Kingham, R. I. and Kallas, B. F., "Laboratory Fatigue and Its Relationship to Pavement Performance," Research Report 72-3, Apr 1972, The Asphalt Institute, College Park, Md.

33. Barker, W. R., "Nonlinear Finite Element Analysis of Heavily Loaded Airfield Pavement Systems," Proceedings, Symposium on Applications of the Finite Element Method in Geotechnical Engineering, May 1972, Vicksburg, Miss.

34. Chou, Y. T., Hutchinson, R. L., and Ulery, H. H., Jr., "A Design Method for Flexible Airfield Pavements," Transportation Research Record 521, Jan 1974, Transportation Research Board, Washington, D. C.

35. Kirwan, R. W., Glynn, T. E., and Bonner, G. A., "The Significance of Material Properties in Flexible Pavement Analysis," Jun 1973, Trinity College, Dublin.

36. Barker, W. R., Brabston, W. N., and Townsend, F. C., "An Investigation of the Structural Properties of Stabilized Layers in Flexible Pavement Systems," Miscellaneous Paper S-73-69, Oct 1973, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

37. Green, J. L. and Hall, J. W., Jr., "Nondestructive Vibratory Testing of Airport Pavements; Experimental Test Results and Development of Evaluation Methodology and Procedure," Report No. FAA-RD-73-205, Vol I, Sep 1975, Federal Aviation Administration, Washington, D. C.

38. Ahlvin, R. G. et al., "Multiple-Wheel Heavy Gear Load Pavement Tests; Basic Report," Technical Report S-71-17, Vol I, Nov 1971, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

39. Burns, C. D. et al., "Comparative Performance of Structural Layers in Pavement Systems; Design, Construction, and Behavior Under Traffic of Pavement Test Sections," Report No. FAA-RD-73-198, Vol I, Jun 1974, Federal Aviation Administration, Washington, D. C.

40. Edwards, J. M. and Valkering, C. P., "Structural Design of Asphalt Pavement for Heavy Aircraft," Koninklijke/Shell Laboratorium, Amsterdam.

41. Dorman, G. M. and Metcalf, C. T., "Design Curves for Flexible Pavements Based on Layered System Theory," Highway Research Record No. 71, 1965, Highway Research Board, Washington, D. C.

42. Finn, F. N., Nair, K., and Monismith, C. L., "Applications of Theory in the Design of Asphalt Pavements," Proceedings, Third International Conference on the Structural Design of Asphalt Pavements, 1972, London.

43. Peattie, K. R., "A Fundamental Approach to the Design of Flexible Pavements," Proceedings, International Conference on the Structural Design of Asphalt Pavements, 1962, Ann Arbor, Mich.

44. Kaplar, C. W., "Freezing Test for Evaluating Relative Frost Susceptibility of Various Soils," Technical Report 250, Jun 1974, U. S. Army Cold Regions Research and Engineering Laboratory, CE, Hanover, N. H.

45. Office, Chief of Engineers, Department of the Army, "Laboratory Soils Testing," Engineering Manual EM 1110-2-1906, Nov 1970, Washington, D. C.

46. Leary, R. M. et al., "Freezing Tests of Granular Material," Highway Research Record No. 215, 1968, Highway Research Board, Washington, D. C.

47. Zoller, J. H., "Frost Susceptibility of New Hampshire Base Courses," Report No. 1972, 1973, Department of Civil Engineering, University of New Hampshire, Durham, N. H.

48. Office, Chief of Engineers, Department of the Army, "Soil Sampling," Engineering Manual EM 1110-2-1907, Mar 1972, Washington, D. C.

49. American Society for Testing and Materials, 1974 Annual Book of ASTM Standards, Part 14, 1974, Philadelphia, Pa.

50. ________, 1974 Annual Book of ASTM Standards, Part 19, 1974, Philadelphia, Pa.

GPO 898-374