AN INDIRECT TENSILE TEST FOR STABILIZED MATERIALS

by

W. Ronald Hudson
Thomas W. Kennedy

Research Report Number 98-1

Evaluation of Tensile Properties of Subbases
for Use in New Rigid Pavement Design

Research Project 3-8-66-98

conducted for

The Texas Highway Department

in cooperation with the
U. S. Department of Transportation
Federal Highway Administration
Bureau of Public Roads

by the

CENTER FOR HIGHWAY RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

January 1968

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Bureau of Public Roads.

PREFACE

This is the first in a series of reports which will be written covering the findings of this research project. This report is intended to summarize the current status of knowledge concerning the indirect tensile test, to report the findings of a limited experimental evaluation of the test for the study of asphalt-stabilized and cement-treated materials, and to describe the equipment and testing technique developed from both the literature review and the experimental evaluation of the test.

Future reports are planned which will be concerned with the tensile characteristics and behavior of asphalt-stabilized, cement-treated, and lime-treated materials. Reports will be written on subjects such as (1) factors affecting the tensile characteristics and behavior of all three materials when subjected to static loads and dynamic repeated loads, (2) correlation of indirect tensile test parameters with parameters from standard Texas Highway Department Tests, and (3) performance criteria for stabilized materials. A final report is planned to summarize all phases of the project.

W. Ronald Hudson
Thomas W. Kennedy

January 2, 1968

This page replaces an intentionally blank page in the original.
-- CTR Library Digitization Team

ABSTRACT

The importance of the tensile characteristics of the subbase of rigid pavements can be demonstrated both from theoretical considerations and from field observations. Information on the tensile behavior and properties of treated and untreated subbase material is limited primarily because of the lack of a satisfactory tensile test. On the basis of a literature review concerned with tensile testing it was concluded that of the currently available tensile tests the indirect tensile test has the greatest potential for the evaluation of the tensile properties of highway materials.

Previous use of the test, unfortunately, has been for evaluating the tensile strength of portland cement concrete and mortar. A limited amount of work has been done on asphaltic concrete and lime-treated soil; however, almost no evaluation of the deformation characteristics has been attempted. Thus, it was felt that the indirect tensile test should be evaluated both theoretically and experimentally before it was used extensively for evaluating the strength and deformation characteristics of stabilized materials.

This report discusses the types of tensile tests, the theory of the indirect tensile test, and factors affecting the indirect tensile test. In addition, the results of a limited testing program to evaluate and develop equipment and a testing technique for the indirect tensile test are discussed. Tentative recommendations are made to the Texas Highway Department with reference to the use of the indirect tensile test and to the technique and method of conducting the test.

This page replaces an intentionally blank page in the original.
-- CTR Library Digitization Team

TABLE OF CONTENTS

NOMENCLATURE

Symbol	Definition
d	Diameter of a disk or indirect tensile test specimen
P	Total load applied to disk or specimen
P'	Load per unit thickness = P/t
P_{max}	Maximum total load applied to disk or specimen
r	Radial distance from point of load application to any element
R	Radius of a disk or indirect tensile test specimen
t	Thickness of a disk or height of an indirect tensile test specimen
S_T	Indirect tensile strength = $\frac{2P_{max}}{\pi td}$
x	Rectangular coordinate perpendicular to the direction of the applied load measured from the center of a disk or indirect tensile test specimen
y	Rectangular coordinate parallel to the direction of the applied load measured from the center of a disk or indirect tensile test specimen
θ	Angle between the direction of the applied load and r
σ_r	Radial stress from the point of load application in the direction of r
σ_t	Indirect tensile stress
σ_x	Stress perpendicular to the direction of the applied load
σ_y	Stress in the direction of the applied load
σ_{yn}	Normal stress on the horizontal diametral section in the direction of the applied load
τ_{xy}	Shearing stress with respect to rectangular coordinates

This page replaces an intentionally blank page in the original.
-- CTR Library Digitization Team

CHAPTER 1. A RATIONALE FOR THE STUDY OF TENSILE PROPERTIES

The design and performance of subbases is a continuing concern of pavement designers. The basic pavement design methods involve either an analysis of a slab-on-foundation or an analysis of stresses in an elastic mass or layered system. The first method, which involves an analysis similar to that developed by Westergaard (Refs 1 and 2) or that of Hudson and Matlock (Refs 3 and 4), considers the subbase as a Winkler foundation consisting essentially of a bed of elastic springs. The second assumes that the base or subbase acts as an elastic layer perfectly bonded to the adjacent layers of pavement. Undoubtedly neither of these analytical techniques represents the real case exactly. These techniques, however, do indicate that tensile stresses are developed in each layer.

Layered analyses predict large tensile stresses in the bottom of each layer directly under the load (Refs 5, 6, and 7). For example, tensile stresses of 50 to 150 psi are predicted in the bottom fibers of the subbase layer of a two-layered elastic system lying on an infinitely thick, medium soft subgrade (Ref 5, p 70).

An analysis of the subbase of a rigid pavement cannot be made using Westergaard methods, but an approximate analysis can be made in two parts using the SLAB analysis techniques developed by Hudson and Matlock for the Texas Highway Department in Research Project No. 3-5-63-56, "Development of Methods for Computer Simulation of Beam-Columns and Grid-Beam and Slab Systems" (Ref 3). Consideration of the 10-inch-thick slab-on-foundation in Fig 1 shows that the deflection patterns are obtained from the solution of the slab on a Winkler foundation. These deflection values can be multiplied by the slab support values to obtain load inputs for an analysis of the subbase layer. Assuming a 9-inch-thick subbase layer with a modulus of elasticity E of 1×10^5 psi, a Poisson's ratio of 0.15, and a subgrade support modulus beneath the subbase layer of 50 lbs per cubic inch, it is possible to calculate the deflections and stresses in the subbase layer. The resulting deflection pattern of the subbase layer is shown in Fig 2, and the

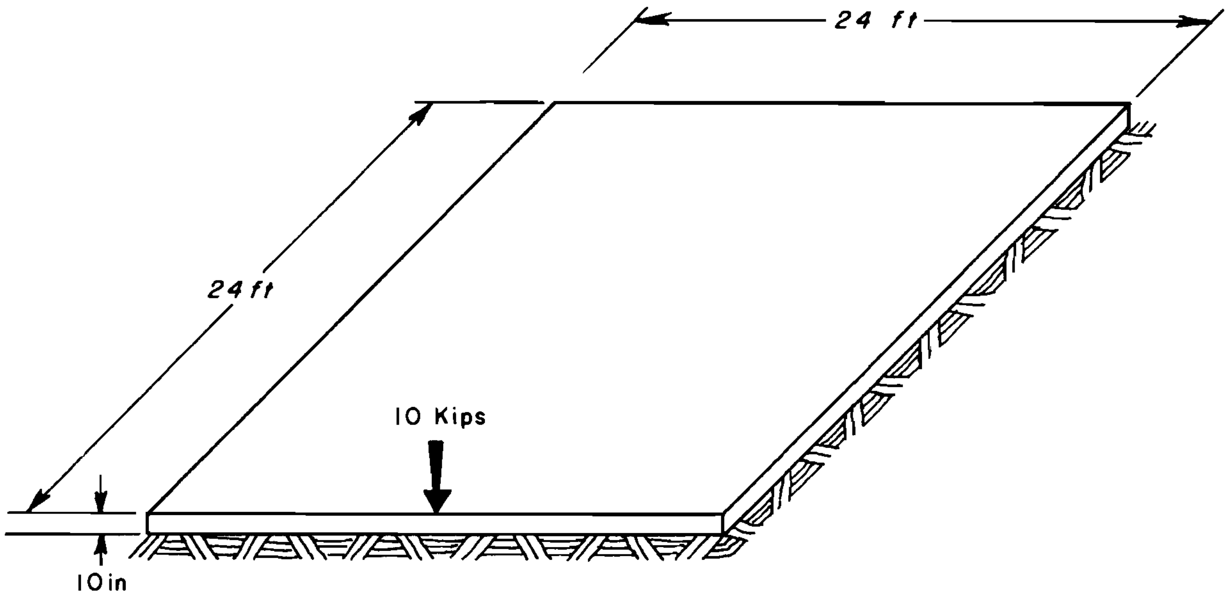


Fig 1. Pavement slab subjected to 10-kip load at the edge with uniform subgrade support.

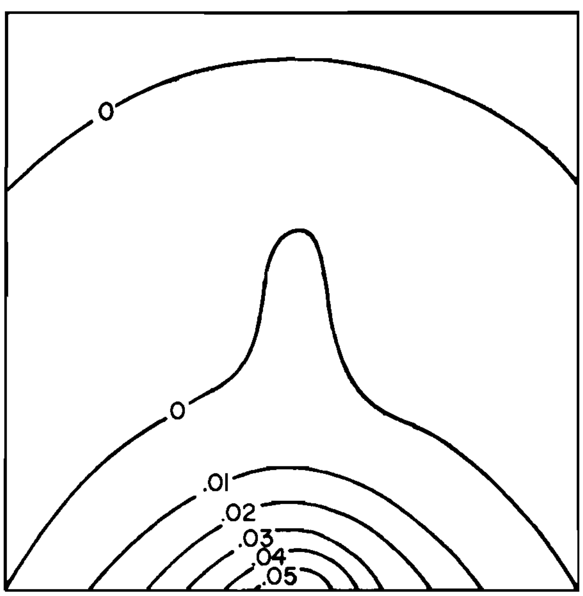


Fig 2. Deflection contours in the subbase layer (inches).

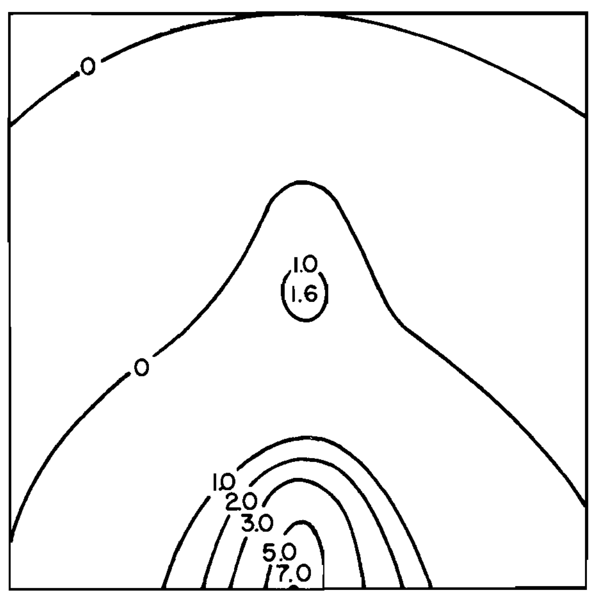


Fig 3. Tensile stress contours in the subbase layer (psi).

stresses in the bottom fibers of the subbase layer are shown in Fig 3. Note that the tensile stresses in the bottom fibers of the subbase layer range up to 7 psi. This is by no means an exact analysis of the problem but is intended to be indicative of the type of stress which can be present in stabilized subbases. For very accurate solutions an iterative process of matching the deflections of the subbase to those of the slab at all points is essential. Such techniques are presently being studied in Research Project No. 3-5-63-56.

The observation and analysis of data from various field sources also indicate the desirability of using stabilized materials for subbase layers. Good examples can be observed in the AASHO Road Test data (Ref 8). Although there were no stabilized subbases under the portland cement concrete pavements, there was a variety of bases in the asphalt concrete studies. Some results of these studies are shown in Figs 4 and 5 and Table 1. In Fig 4 note that the thickness of untreated base material required to maintain satisfactory performance for the life of the test was nearly three times as great as the required thickness of bituminous-treated materials, regardless of the load involved. Figure 5 shows the same general effect for 18-kip axle-loads regardless of the number of applications involved. The cement-treated sections performed much better than the crushed stone bases, but were less effective than the bituminous-treated sections. In all cases, the performance of stabilized materials was significantly better than that of unstabilized materials. As shown in Table 1, the crushed stone, which had only slight tensile strength, also performed better than the gravel alone, which had no tensile strength. An extensive study of these materials is planned for later stages of this project.

The Road Test portland cement concrete pavements were designed with a so-called "nonpumping subbase," a well-graded sand-gravel with maximum-sized stone of 1-1/2 inches. The results of the Test show extensive pumping of this material (Figs 6 and 7). While there were no stabilized subbase materials with which to compare this gravel subbase, it seems highly desirable to evaluate the characteristics of better quality materials for use as subbases. Future field tests of the Road Test type should definitely include some sections of stabilized subbase.

The Research and Development Laboratory of the Portland Cement Association has long been interested in the effect of stabilized bases on performance

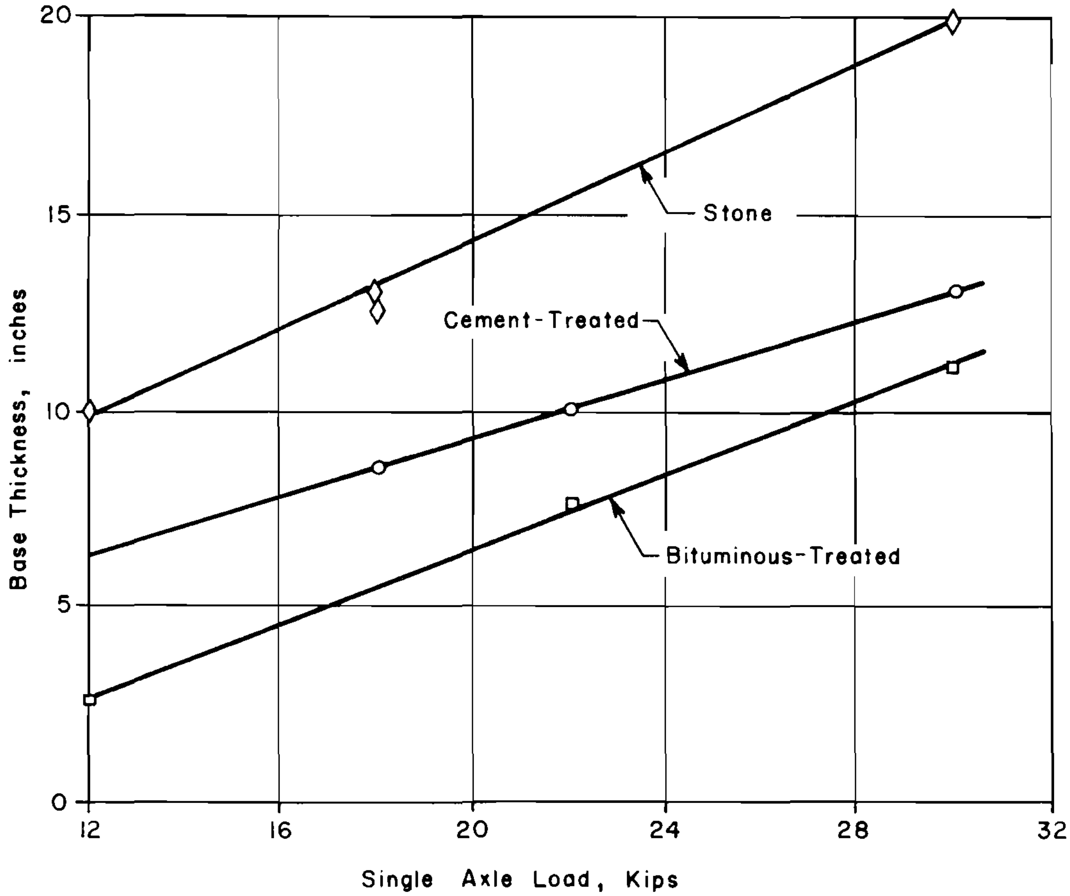


Fig 4. Base thickness required to maintain satisfactory performance for 1,114,000 load applications (Ref 8).

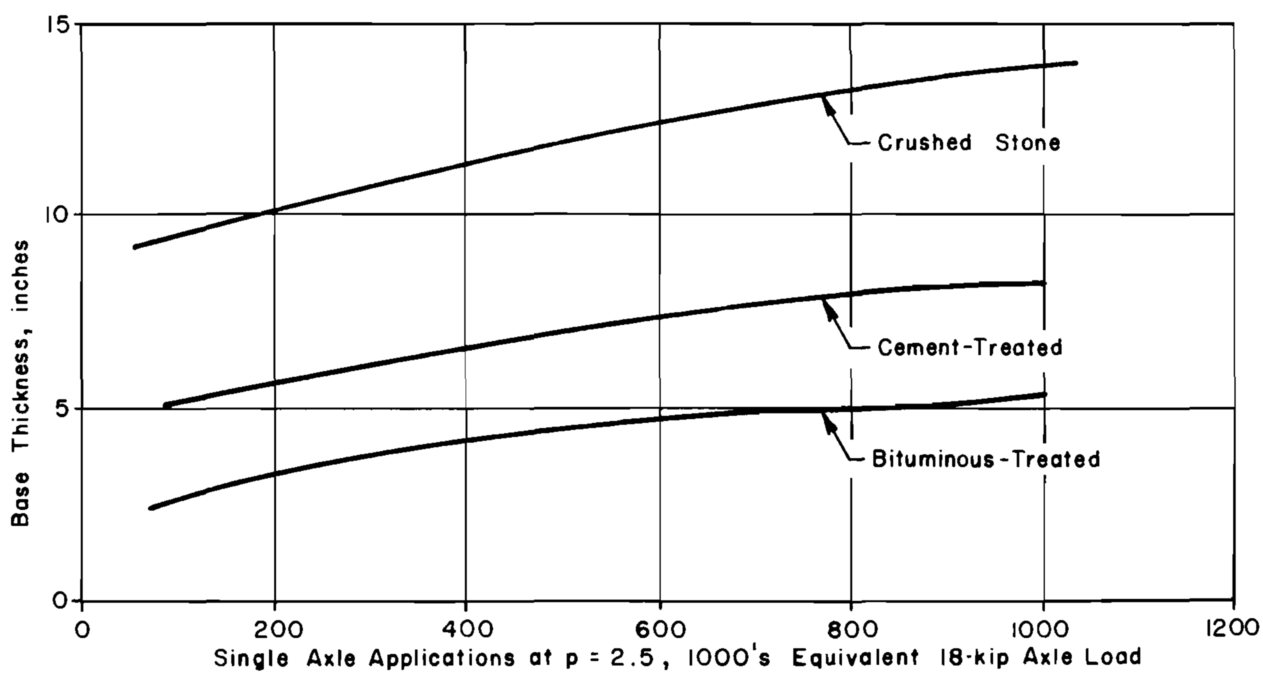


Fig 5. Relationship between base thickness and single axle-load applications (Ref 8).

TABLE 1. THICKNESS OF BASE WHICH DID NOT CRACK UNDER THE SPECIFIED NUMBER OF APPLICATIONS OF LOAD (REF 8)*

Base Type	Number of Applications	Single Axle Load Applied In Kips 12	18	22.4	30
Gravel	500,000	9.3 in.**	OT***	OT***	--
		9.5 in.**	OT***	OT***	--
	1,114,000	OT***	OT***	OT***	--
		OT***	OT***	OT***	--
Stone	500,000	7.7 in.	9.2 in.	--	11.0 in.
		7.7 in.	9.2 in.	--	11.0 in.
	1,114,000	11.4 in.	11.0 in.	--	11.0 in.
		13.0 in.	13.2 in.	--	11.2 in.
Cement	500,000	--	6.2 in.	6.9 in.	9.1 in.
		--	6.3 in.	7.0 in.	9.1 in.
	1,114,000	--	7.8 in.	8.0 in.	9.5 in.
		--	7.5 in.	8.8 in.	10.2 in.
Bituminous	500,000	2.4 in.	--	5.3 in.	6.7 in.
		2.6 in.	--	5.3 in.	6.7 in.
	1,114,000	2.6 in.	--	5.3 in.	7.6 in.
		2.8 in.	--	6.9 in.	8.0 in.

*This data is taken from the AASHO Road Test, special base experiment reported in Ref 8, p 57.

**Double values indicate that there were two sections tested.

***Out of Test - Indicates that the section failed at a reduced number of applications.

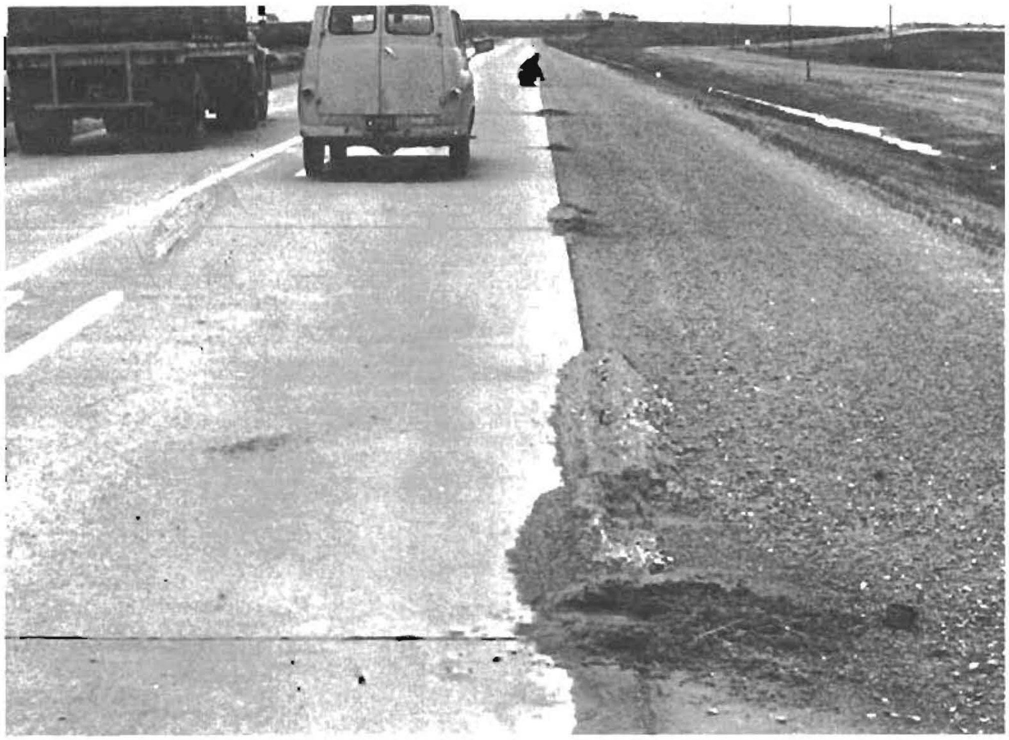


Fig 6. Typical quantities of granular subbase material (maximum size $1\frac{1}{2}$") pumping from beneath portland cement concrete panel at the AASHO Road Test.

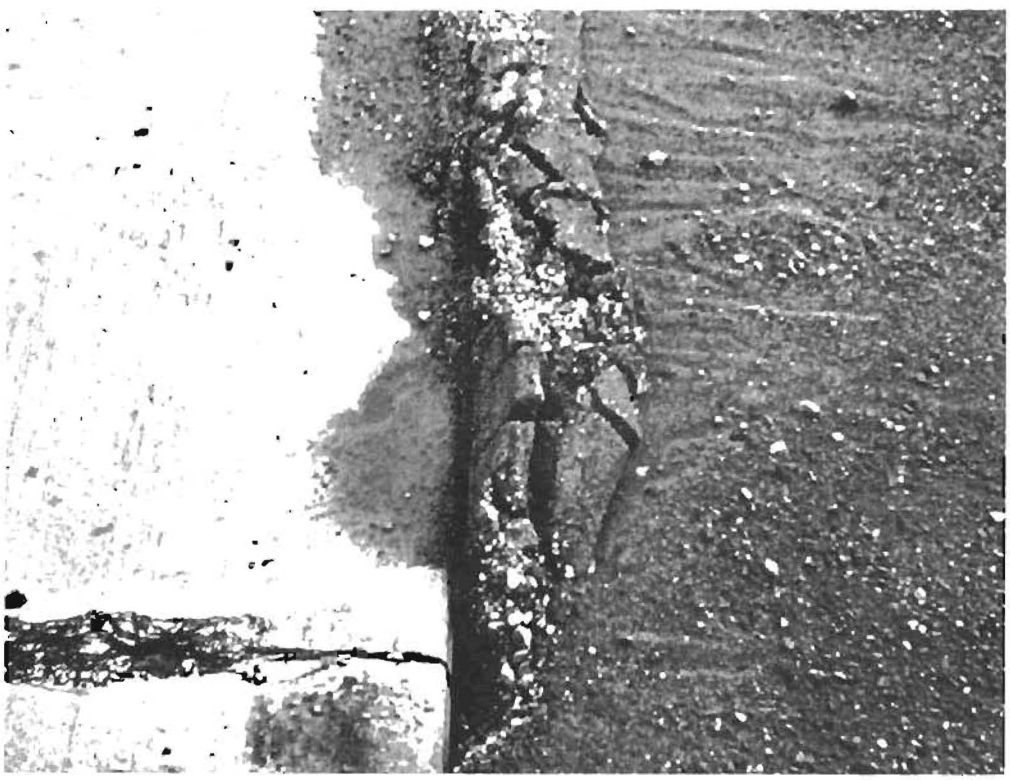


Fig 7. Subbase material ejected from beneath pavement at the AASHO Road Test after a heavy rain.

of concrete pavement. In particular they have studied cement-treated bases. The results of these studies are well documented (Refs 9 and 10). In recent studies the tensile stresses in the subgrade were indicated to be well above the modulus of rupture of the cement-treated subbase.

Documented observations of stabilized subbases under portland cement concrete pavements in Texas are sparse. In his paper at the Fortieth Annual Texas Highway Shortcourse, R. S. Williamson (Ref 11) reports on several cases of subbase pumping involving unstabilized gravel and lime-stabilized clay. In these cases the tensile strength or cohesion of the materials was inadequate to prevent pumping. No evidence of adverse performance, however, has been reported in Texas for pavements with high-quality stabilized subbase materials.

Thus, experience from several sources indicates the benefits derived from the use of stabilized materials in pavement construction, yet little is known about the behavior and design of subbase materials. From such evidence it is logical to assume that the cohesive or tensile characteristics of the subbase significantly affect pavement performance. Unfortunately, little information is available on the tensile behavior and properties of treated and untreated subbase material. A primary reason has been the lack of a satisfactory tensile test. The purpose of this report is to evaluate tensile testing and to establish a tentative recommended tensile test and a testing procedure.

This page replaces an intentionally blank page in the original.
-- CTR Library Digitization Team

CHAPTER 2. TYPES OF TENSILE TESTS FOR HIGHWAY MATERIALS

Various tests and modifications have been developed and used for evaluating the tensile characteristics of highway materials. These tests can be classified as (1) direct tensile tests, (2) bending tests, or (3) indirect tensile tests. (Each of these tests is discussed and analyzed in this report.)

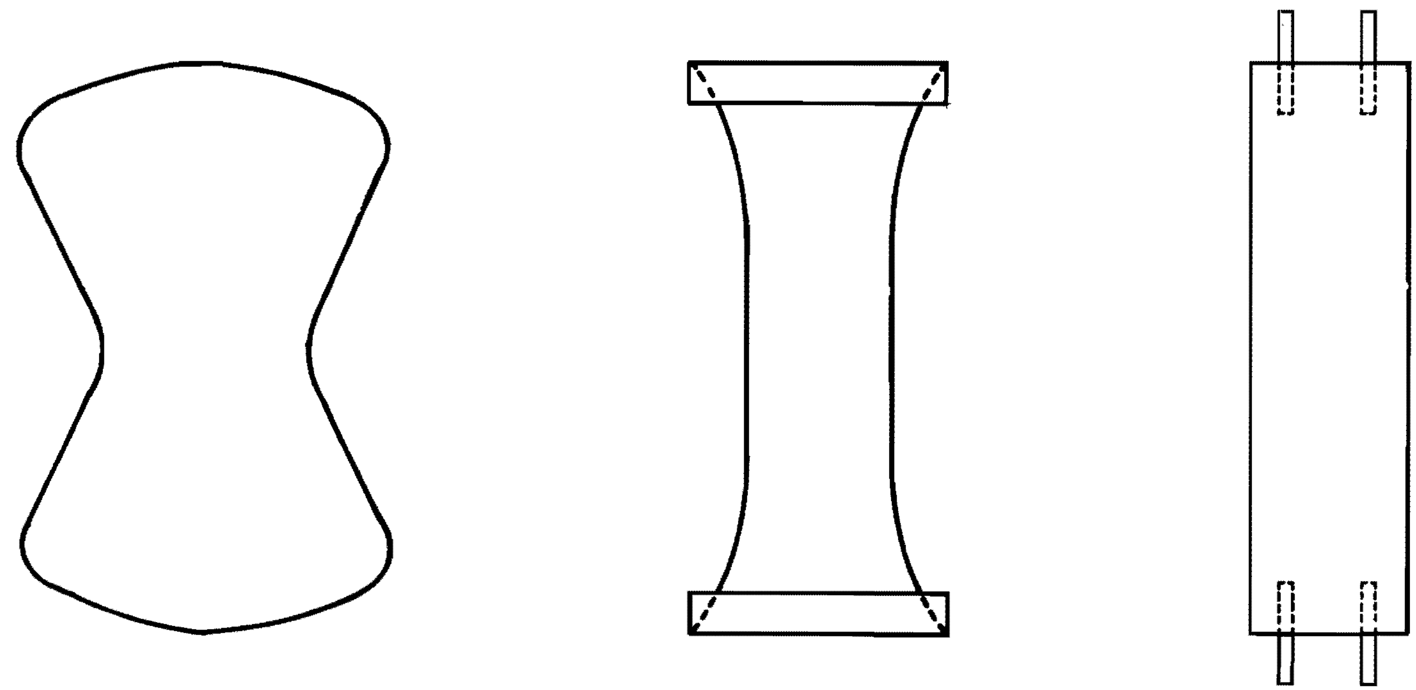
DIRECT TENSILE TEST

The direct tensile test is simple in theory and principle. It consists of applying an axial tensile force directly to a specimen and measuring the stress-strain characteristics of the material.

The primary variations in the test are in the size and shape of the specimen and the methods of gripping the tensile specimen. A few of the possible specimen shapes used are shown in Fig 8. Another modification involves the orientation of the specimen. Messina (Ref 12) and others in studies of asphaltic concrete have conducted direct tensile tests by applying the load through semicircular loading heads which were cemented to the periphery of a cylindrical specimen (Fig 9). This testing technique was chosen in an attempt to reduce the effect of the planes of weakness produced by compaction in layers and because it seems to offer a convenient method of attaching grips to the specimen.

Although such tests seem simple, serious difficulties have been encountered in practical applications. The major problems have included the addition of bending stresses due to alignment problems and the gripping of the specimen.

In analyzing the test is is assumed that only pure tension is applied to the specimen. Any eccentricity or misalignment of the applied load will result in bending stresses which introduce errors in the test results. Although this problem is more serious in brittle materials than in ductile materials, which can relieve these bending stresses by plastic flow, it nevertheless has been found that the application of a pure tensile force is a difficult and time-consuming task.



(a) Briquet (square cross section at middle).

(b) Bobbin (circular section).

(c) Cylinder or prism with embedded studs.

Fig 8. Different types of tensile specimens (Ref 14).

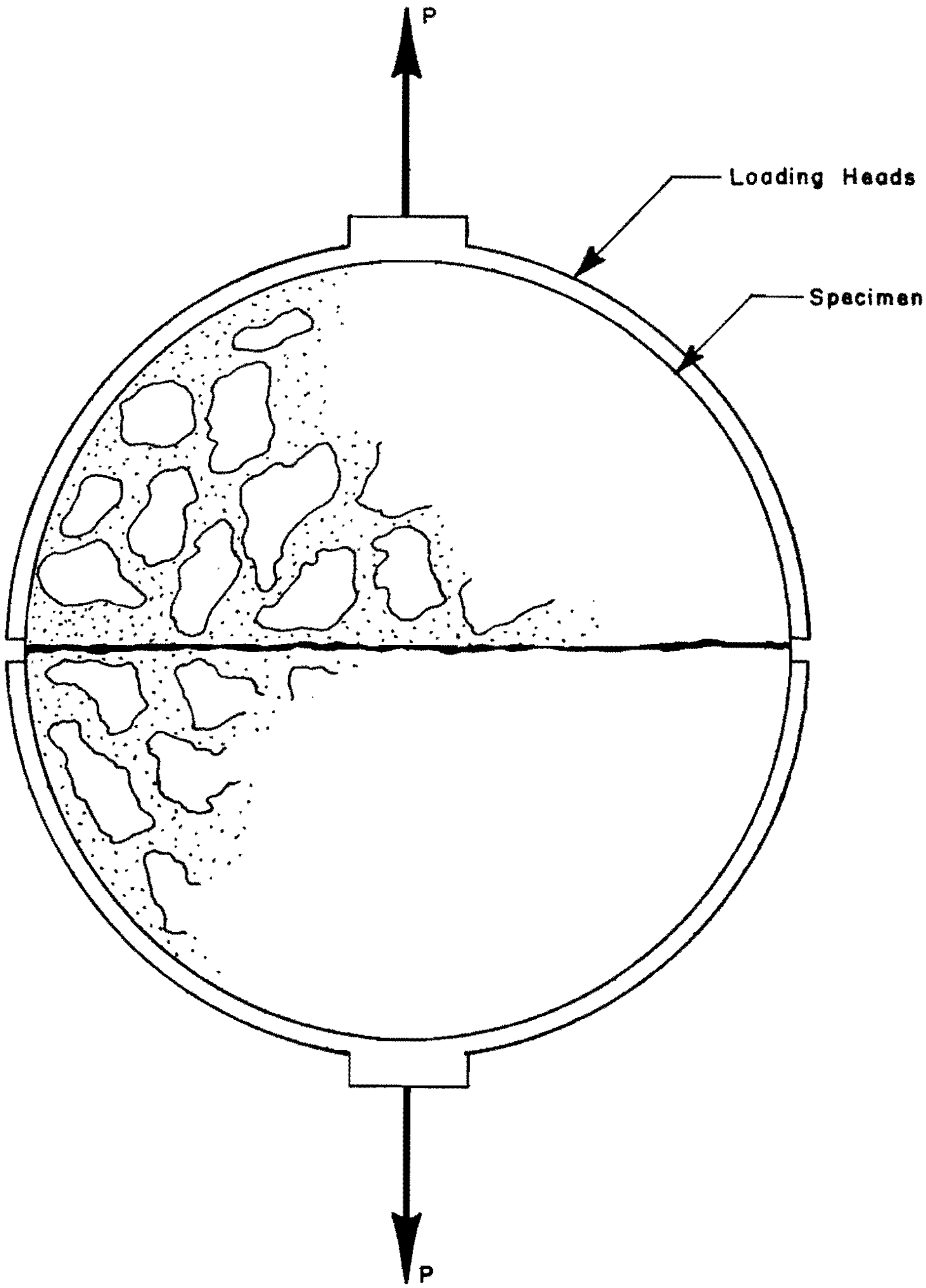


Fig 9. Diametrical direct tensile test (Ref 12).

The second major source of difficulty is associated with the method of gripping the tensile specimen. Mitchell (Ref 13) reports that photoelastic studies show large stress concentrations at the loading grips on a figure-eight briquet. Various shapes and methods of gripping the specimen have been used in an attempt to alleviate this problem of secondary stress, and the use of epoxy for attaching the loading head to the specimen has probably reduced the problem. It should be noted, however, that the fabrication of these specimens is complicated and requires great care. This test is probably not applicable to brittle materials or to materials containing large aggregate sizes. Brittle materials cannot relieve stress concentrations and large aggregate particles make it difficult to produce specimens free of surface irregularities.

Another problem associated with the test concerns the evaluation of the test results. Engineers normally assume that the stress is distributed uniformly across the cross section, but Mitchell (Ref 13) reports that the maximum stress on the central cross section of a figure-eight briquet is about 1.75 times the average stress. In view of these difficulties and uncertainties it is felt that the direct tensile test has limited application and that test results obtained by this method are questionable.

BENDING TESTS

The second category of tensile tests, bending tests, involves the application of a bending load to a beam specimen. This test is considerably simpler to conduct than the direct tensile test and requires less care in the preparation of the specimens. It is favored by many engineers because the loading conditions are similar to the field loading condition of pavement materials. Basically this test involves two types of loading conditions. The common flexure test is conducted by applying a load to a simply-supported beam (Fig 10), while the cohesiometer test involves the application of a bending moment to a specimen through a cantilever arm (Fig 11).

There are two standard methods for applying load to a simply-supported beam. The load may be applied as two equal, concentrated loads at the third points (Ref 14) of the beam or as a single concentrated load at the midpoint (Ref 15) of the beam (Fig 10). The strength parameter is normally expressed by the modulus of rupture or by relating the modulus of rupture to the tensile strength.

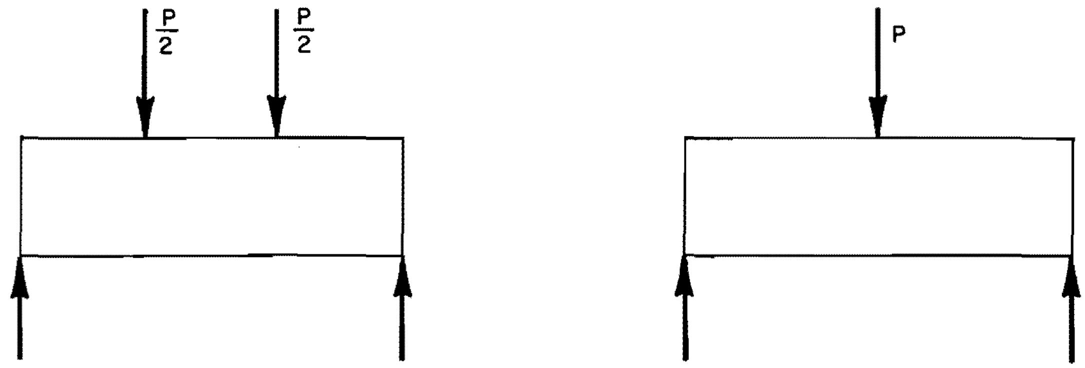


Fig 10. Loading conditions for determination of modulus of rupture.

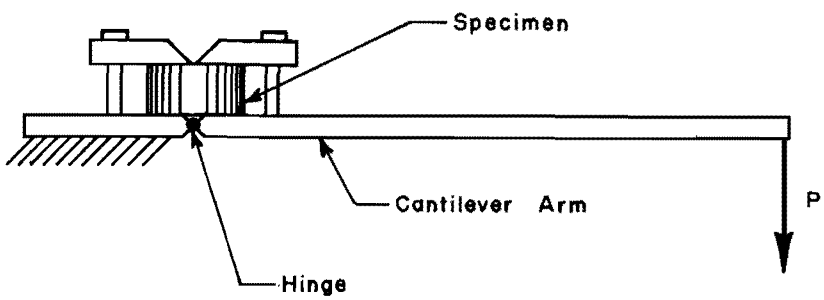


Fig 11. Cohesiometer test.

The modulus of rupture is calculated by the standard flexure formula using the dimensions of the beam and the applied bending moment at the point at which the beam fails. This formula, however, assumes that stress is proportional to the distance from the neutral axis which in turn is dependent on a linear stress-strain relationship for the material tested. Such a relationship does not exist for most materials. More important is the fact that even in the more elastic materials this assumption is seriously in error at failure conditions. The net effect usually produces a modulus of rupture which is much higher than the actual failure stress. Grieb and Werner (Ref 16) and Thaulow (Ref 17) estimate that for concrete the modulus of rupture is equal to or greater than two times the tensile strength. One method of utilizing the modulus of rupture is to consider it to be an index of tensile strength. A second method is to establish a relationship between the modulus of rupture and the tensile strength. According to Mitchell (Ref 13) the latter approach has not been too satisfactory since the relationship has generally been assumed to be linear when in reality it appears to be curvilinear.

The cohesiometer test was developed by the California Highway Department as a means of evaluating the cohesive resistance or tensile characteristics of highway materials. The test consists of clamping a sample in the testing device directly over a hinge (Fig 11). One end of the specimen is held fixed and the other is loaded through a cantilever arm, producing failure. The load required to cause failure is used to calculate the cohesiometer value (grams per inch of width corrected to a 3-inch height). This value is empirical and has no theoretical counterpart.

Unfortunately, there appears to be a lack of information concerning the initial theory of the cohesiometer test (Ref 18). Basically, however, the specimen is required to supply an internal moment to resist the applied moment. This internal resisting moment is dependent on the tensile stress developed in the specimen.

The major criticisms of both types of bending tests concern the nonuniform and undefined stress distribution which exists across the specimen and the fact that the maximum tensile stress occurs at the outer surface. This latter condition accentuates the effect of surface irregularities and may result in low indicated values of tensile strength.

INDIRECT TENSILE TEST

The indirect tensile test was developed simultaneously but independently by Carneiro and Barcellos (Ref 19) in Brazil and Akazawa (Ref 20) in Japan The test involves loading a cylindrical specimen with compressive loads distributed along two opposite generators (Fig 12). This condition results in a relatively uniform tensile stress perpendicular to and along the diametral plane containing the applied load. The failure usually occurs by splitting along this loaded plane.

Previous use of this test has generally been on concrete or mortar specimens; however, Thompson (Ref 21) found the test to be satisfactory for the evaluation of the tensile characteristics of lime-soil mixtures while Messina (Ref 12) and Breen and Stephens (Refs 22 and 23) used the test for the study of asphaltic concrete. In addition, Livneh and Shklarsky (Ref 24) used the test in the evaluation of anisotropic cohesion of asphaltic concrete. From a review of the literature concerned with the evaluation and use of the indirect tensile test a number of advantages and one major disadvantage were found. The main disadvantage is the fact that the loading conditions of the test do not resemble those in the field. The six major advantages attributed to the test are that

(1) It is relatively simple.

(2) The type of specimen and equipment are the same as that used for compression testing.

(3) Failure is not seriously affected by surface conditions.

(4) Failure is initiated in a region of relatively uniform tensile stress.

(5). The coefficient of variation of the test results is low.

(6) Mohr's theory is a satisfactory means of expressing failure conditions for brittle crystalline materials such as concrete (Ref 13).

CHOICE OF TEST

On the basis of the literature review concerned with tensile testing it was concluded that of the currently available tensile tests the indirect tensile test has the greatest potential for the evaluation of the tensile properties of highway materials. As previously noted the only major disadvantage attributed to the test concerns its failure to duplicate field loading conditions. While such a loading condition may be desirable, its lack is not

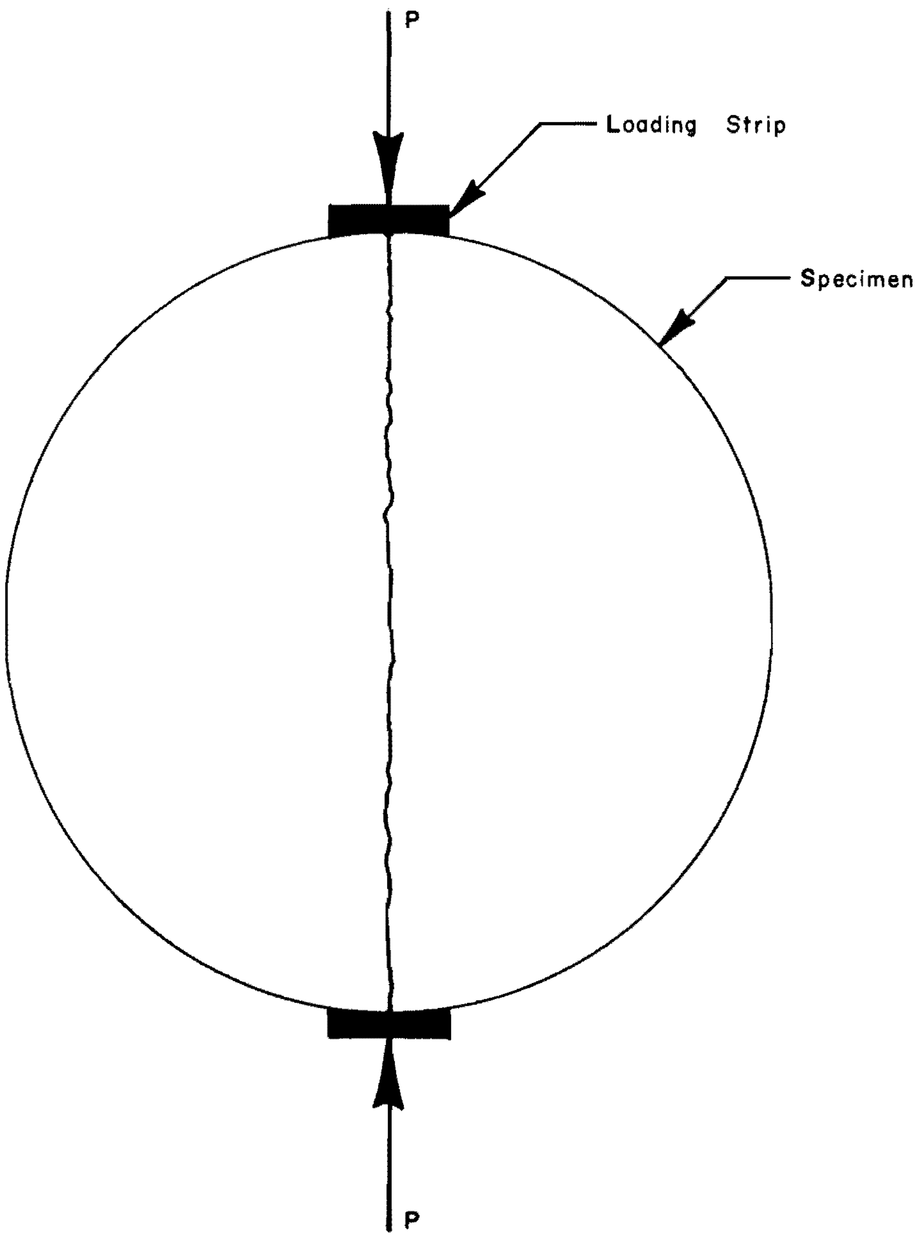


Fig 12. The indirect tensile test.

decisive and is more than offset by the many apparent advantages of the test. A secondary disadvantage of the test is that the theory is more complex than for direct tensile and bending tests. Once again the many advantages of the test seem to more than offset the increased complexity of the theory upon which the test is based. Thus, the indirect tensile test has been given priority for use in this project in evaluating the tensile properties of stabilized highway materials. Also, it is recommended that other agencies consider this test as a means of investigating and evaluating highway materials.

This page replaces an intentionally blank page in the original.
-- CTR Library Digitization Team

CHAPTER 3. THEORY OF INDIRECT TENSILE TEST

According to Thaulow (Ref 17) the theory for the stress distribution for the indirect tensile test was first developed by Hertz (Ref 25). Later A. Foppl and L. Foppl (Ref 26), Timoshenko and Goodier (Refs 27 and 28), Frocht (Ref 29) and Peltier (Ref 30) considered the theory.

TIMOSHENKO'S DEVELOPMENT

Timoshenko and Goodier (Ref 27) consider the case of a concentrated force P at a point A on a line along a straight boundary of an infinitely large plate (Fig 13). The load is assumed to be distributed uniformly over a unit thickness so that P' is the load per unit thickness. This loading condition results in a simple radial distribution of stress so that any element C at a distance r from the point of load application is subjected to compression in the radial direction. The magnitude of this radial stress is

$$\sigma_r = \frac{-2P' \cos\theta}{\pi r} \tag{1}$$

where θ is the angle between the direction of the applied load and a line from the point of load application to the element C , $\overline{AC}$.

Timoshenko and Goodier (Ref 28) then extend this concept by considering two equal and opposite forces of magnitude P' to act along the vertical diameter $\overline{AB}$ of a circular disk as shown in Fig 14. Assuming each of these forces produces a simple radial stress distribution, it can be seen that any point M on the circumference is subjected to compressive stresses acting in the directions of r and r_1 . Since r and r_1 are perpendicular to each other it can be shown that these two stresses are principal stresses of magnitude $\frac{2P'}{\pi d}$. This is equivalent to a hydrostatic stress condition. Since for a disk the surrounding material has been removed, a normal compressive stress of magnitude $\frac{2P'}{\pi d}$ must be applied to the circumference of the disk in order to maintain the assumed pair of radial stresses. In reality the boundary of the disk is free from external stresses; therefore, the stress at any point in

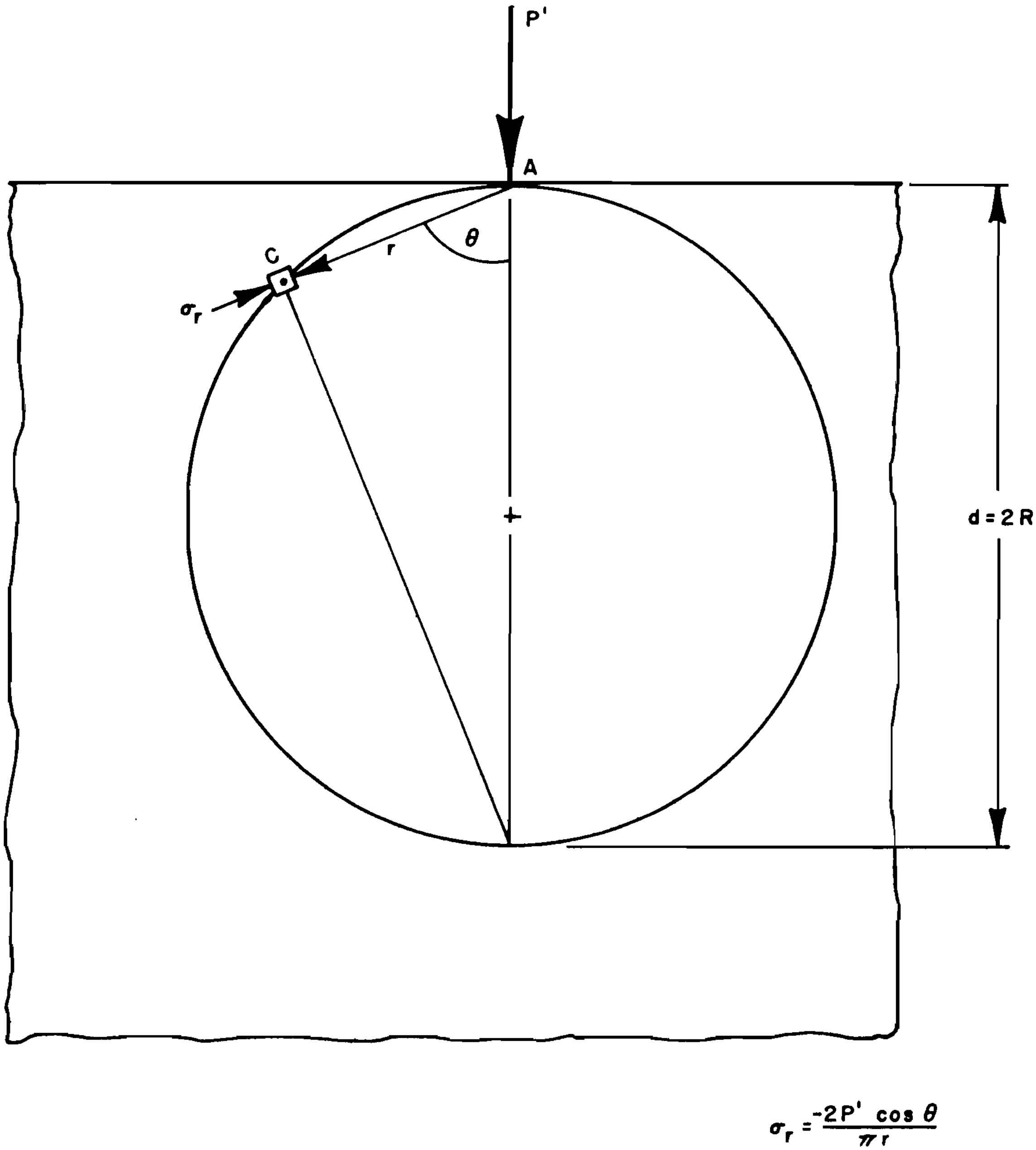


Fig 13. Concentrated force at a point of a straight boundary (Ref 27).

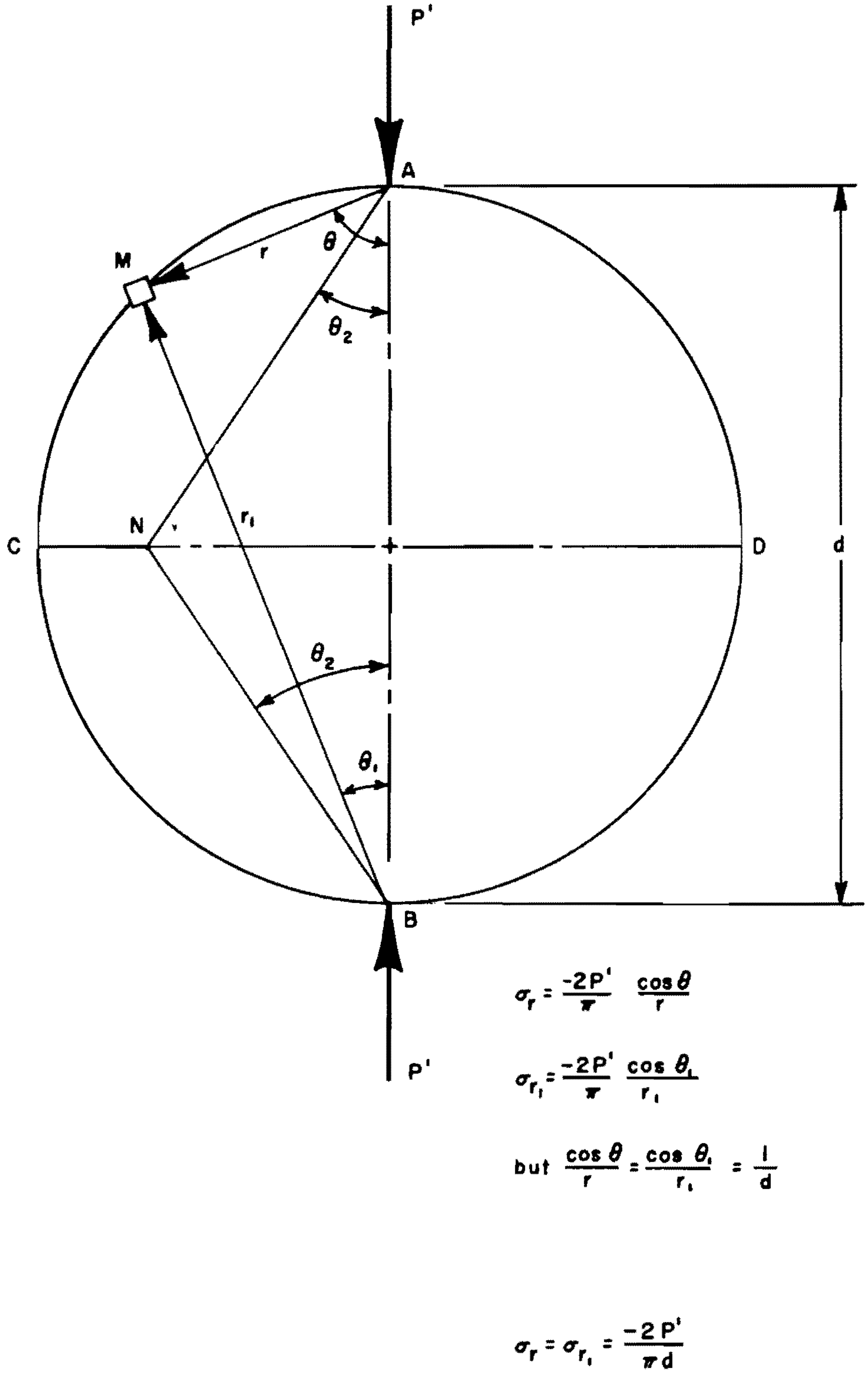


Fig 14. Stresses in a circular disk (Ref 28).

the disk can be obtained by superposing on the system a uniform tensile stress of magnitude $\frac{2P'}{\pi d}$.

Considering the stress on the horizontal diametral section at N , Fig 14, it is concluded from symmetry that there are no shearing stresses on this plane. The normal stress σ_{yn} produced by the two equal radial compressive stresses may be determined from a Mohr's circle analysis as twice the normal stress produced by one load P' or

$$\sigma_{yn} = \frac{-4P'}{\pi} \frac{\cos^3\theta_2}{r} \tag{2}$$

in which r is the distance $\overline{AN}$ and θ_2 is the angle between $\overline{AN}$ and the vertical diameter. Superposing a uniform tensile stress $\frac{2P'}{\pi d}$ yields the following relationship for the normal stress on the horizontal plane at N :

$$\sigma_y = \frac{-2P'}{\pi d}\left[\frac{4d^4}{(d^2 + 4x^2)^2} - 1\right] \tag{3}$$

With regard to the distribution of σ_x along the horizontal diametral section it can similarly be shown that

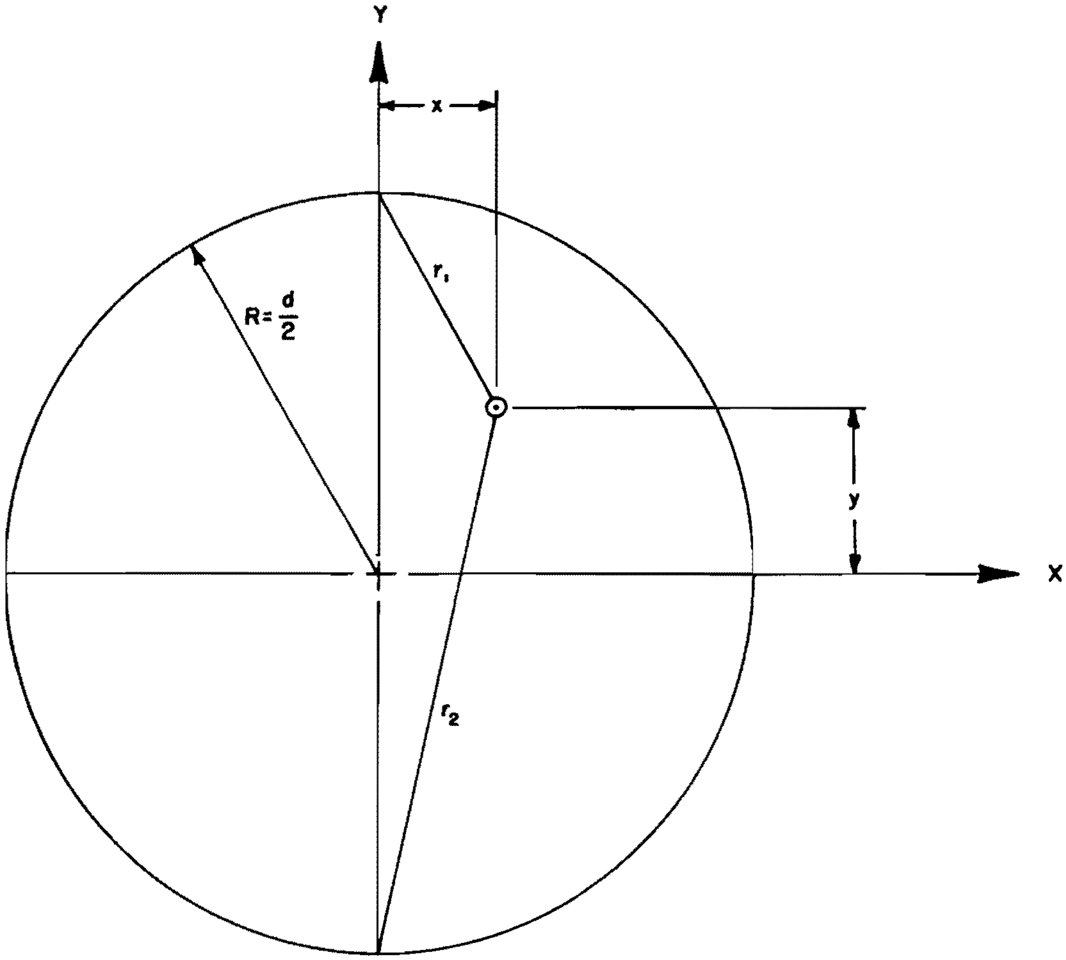
$$\sigma_x = \frac{2P'}{\pi d}\left[\frac{d^2 - 4x^2}{d^2 + 4x^2}\right]^2 \tag{4}$$

FROCHT'S DEVELOPMENT

The above relationships are based on theory of elasticity and describe the stress distributions in the x and y-directions along the horizontal diameter of the specimen. Usually, however, the theory of the indirect tensile test is developed from Frocht's equations for stresses at a point (Ref 29). A brief discussion of this development is included for comparison. Frocht's equations in terms of the rectangular coordinates shown in Fig 15 are

$$\sigma_x = \frac{-2P}{\pi t}\left[\frac{(R - y)x^2}{r_1^4} + \frac{(R + y)x^2}{r_2^4} - \frac{1}{d}\right] \tag{5}$$

$$\sigma_y = \frac{-2P}{\pi t}\left[\frac{(R - y)^3}{r_1^4} + \frac{(R + y)^3}{r_2^4} - \frac{1}{d}\right] \tag{6}$$



$$\sigma_x = \frac{-2P}{\pi t}\left[\frac{(R-y)x^2}{r_1^4} + \frac{(R+y)x^2}{r_2^4} - \frac{1}{d}\right]$$

$$\sigma_y = \frac{-2P}{\pi t}\left[\frac{(R-y)^3}{r_1^4} + \frac{(R+y)^3}{r_2^4} - \frac{1}{d}\right]$$

$$\tau_{xy} = \frac{2P}{\pi t}\left[\frac{(R-y)^2 x}{r_1^4} - \frac{(R+y)^2 x}{r_2^4}\right]$$

Fig 15. Coordinate system and Frocht's equations for stresses at a point (Ref 29).

$$\tau_{xy} = \frac{2P}{\pi t}\left[\frac{(R - y)^2 x}{r_1^4} - \frac{(R + y)^2 x}{r_2^4}\right] \tag{7}$$

where

t = thickness of the disk

and

P = total load applied to the disk or specimen.

For the special case of the horizontal diametral plane of the cylinder, where $y = 0$ and $r_1 = r_2 = (x^2 + R^2)^{\frac{1}{2}}$, the above equations reduce to

$$\sigma_x = \frac{2P}{\pi td}\left[\frac{d^2 - 4x^2}{d^2 + 4x^2}\right]^2 \tag{8}$$

$$\sigma_y = \frac{-2P}{\pi td}\left[\frac{4d^2}{(d^2 + 4x^2)^2} - 1\right] \tag{9}$$

$$\tau_{xy} = 0 . \tag{10}$$

For the special case of the vertical diametral plane through the cylinder, where $x = 0$, $r_1 = R - y$, and $r_2 = R + r$, the above equations reduce to

$$\sigma_x = \frac{2P}{\pi td} = \text{Constant} \tag{11}$$

$$\sigma_y = \frac{-2P}{\pi t}\left[\frac{2}{d - 2y} + \frac{2}{d + 2y} - \frac{1}{d}\right] \tag{12}$$

$$\tau_{xy} = 0 . \tag{13}$$

The distributions of these stresses are shown in Fig 16 for the horizontal diameter and Fig 17 for the vertical diameter. The vertical stress σ_y along the horizontal diameter is compressi and the magnitude varies from a maximum of $\frac{6P}{\pi td}$ at the center to zero at the circumference. The horizontal stress σ_x along the horizontal diameter is tensile with the magnitude varying from a maximum of $\frac{2P}{\pi td}$ at the center to zero at the circumference. The horizontal

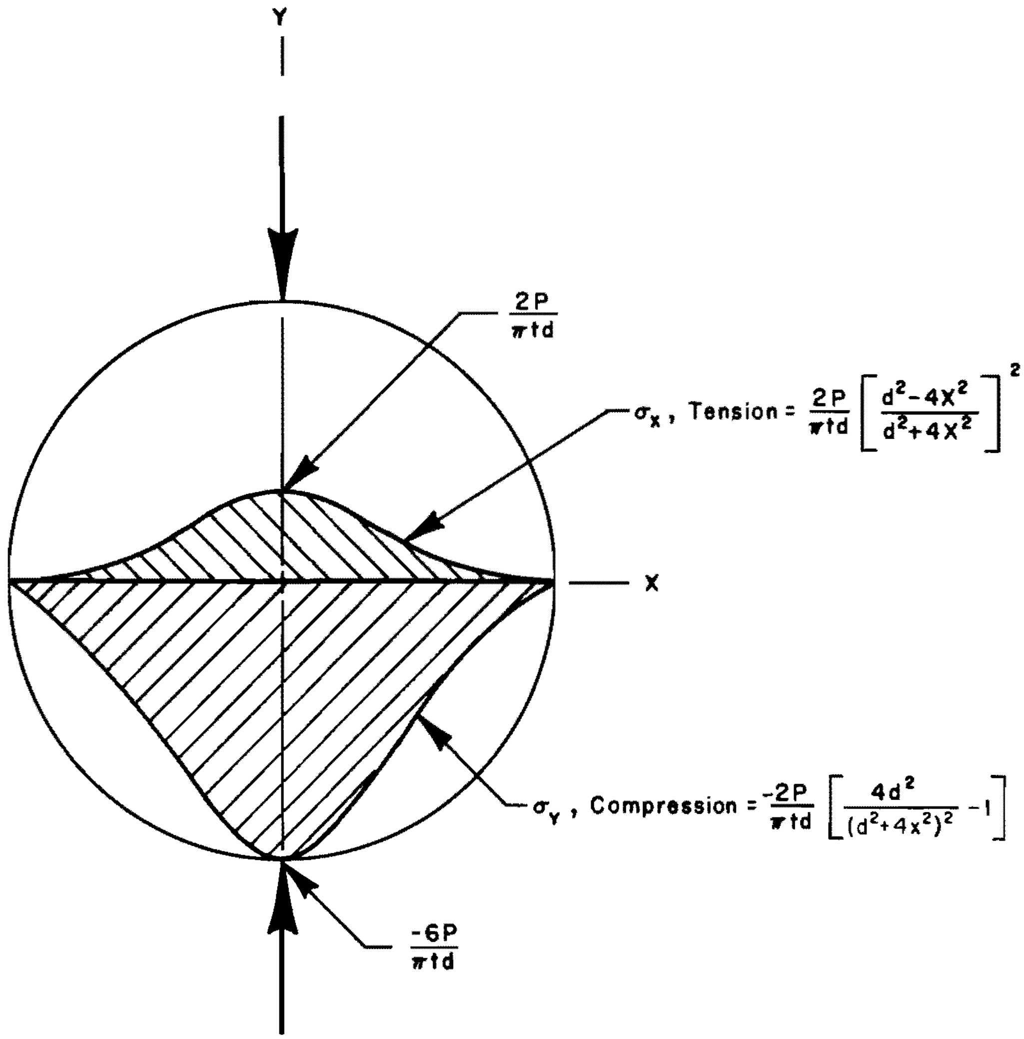


Fig 16. Stress distributions on x-axis.

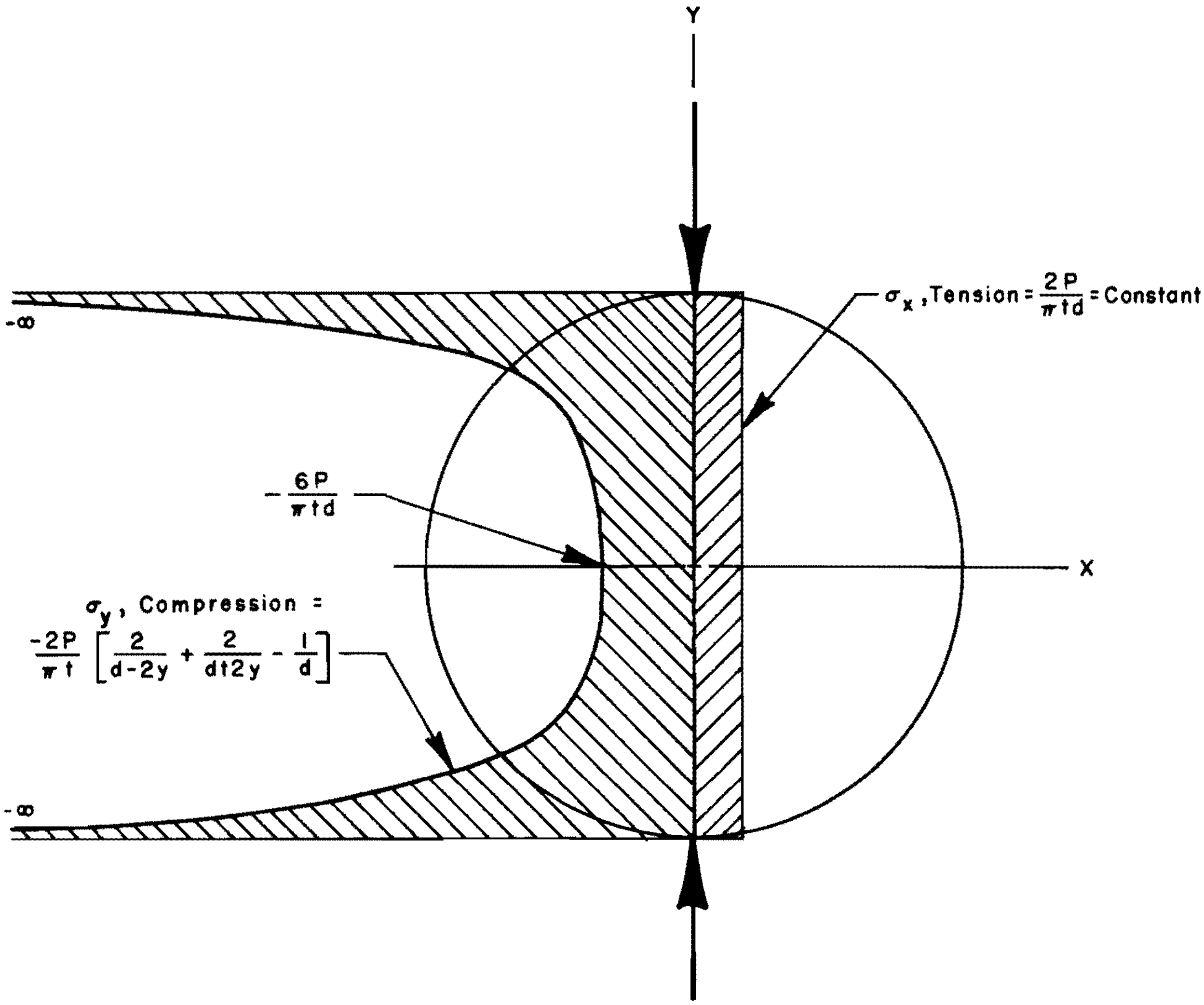


Fig 17. Stress distributions on y-axis.

stress σ_x along the vertical diameter is a constant tensile stress of magnitude $\frac{2P}{\pi td}$ while the vertical stress σ_y is compressive and varies from a minimum of $\frac{6P}{\pi td}$ at the center to a maximum of infinity at the circumference beneath the loads.

Under conditions of a line load, the specimen would be expected to fail near the load points due to compressive stresses and not in the center portion of the specimen due to tensile stress. It has been shown, however, that these compressive stresses are greatly reduced by distributing the load through a loading strip. In addition, the horizontal tensile stress along the vertical diameter changes from tension to compression near the points of load application. These changes are considered in some detail in the following section.

DEVIATION OF TEST FROM IDEAL CONDITIONS

The development described above is an exact solution for the idealized case considered. In reality the actual test deviates from the assumed ideal condition. The following deviations should be considered:

Heterogeneous Nature of Material Tested

The theory on which this test is based assumed a homogeneous material. Stabilized materials are normally heterogeneous not homogeneous; nevertheless, the greatest application of the test has been with concrete which is also very heterogeneous. In addition, Messina (Ref 12) and Livneh and Shklarsky (Ref 24) used the test for the evaluation of asphaltic concrete, a nonhomogeneous material, and Thompson (Ref 21) evaluated lime-soil mixtures with the indrect tensile test. In all of these cases the test was found to be satisfactory although undoubtedly errors were introduced by the heterogeneous nature of the tested materials. With regard to this problem, Wright (Ref 31) concluded that although the effect on the general stress distribution cannot be determined it is probably small enough to permit the use of the test.

Distribution of Applied Load

The theory of the test assumes a point load on a thin disk which corresponds to line loading along a generator of the cylinder. Actually the load is distributed over an area with an appreciable width because of the practice of applying the load through a loading strip.

Rudnick et al (Ref 32) investigated the effects of a load strip on stress distribution through the use of photoelasticity. From this investigation they concluded that the magnitude of the vertical compressive stresses was significantly reduced and that the magnitude of the horizontal stress was virtually unaffected near the center of the specimen but was changed to compression near the edges. Rudnick et al also report that Peltier (Ref 30) calculated the stress distribution in an indirect tensile test specimen for various assumed pressure distributions. His results indicated that the tensile stresses can be held uniform over a reasonable portion of the loaded diameter if the width of the bearing area is less than one-fifth the specimen diameter. Wright (Ref 31) also conducted a theoretical evaluation of the effect of loading strip width on the horizontal stress distribution along the vertical axis. The resulting stress distribution is shown in Fig 18.

Wright's stress distribution is dependent both on the characteristics of the material tested and the characteristics of the loading strip. Mitchell (Ref 13) states that theoretical considerations indicate that both the best type of material and best size of strip will vary as the ratio σ_t/σ_c varies and will depend on the shape of the Mohr failure envelope. Rudnick et al (Ref 32) noted that the length along the loaded diameter over which the horizontal tensile stresses are essentially constant and the magnitude of the stress values outside this constant region are both functions of the mechanical properties of the specimen and the loading strips. If, for example, both the specimen and loading strips have high elastic moduli, the horizontal tensile stress will be constant over a large portion of the loaded diameter but the maximum compressive and shear stresses will be very large and failure may occur in compression or shear. If, however, the loading strip is very soft and the load is applied over too great an area, the horizontal stresses in the center portion of the specimen will be affected.

A number of investigations have indicated that a semisoft material is desirable as a loading strip. Rudnick et al (Ref 32) recommend that the loading strip should be soft enough to allow distribution of the load over a reasonable area and yet narrow enough to prevent the contact area from becoming excessive. The basic requirement or criterion for selection of the loading strip is that it produce tensile rather than compressive or shear failures.

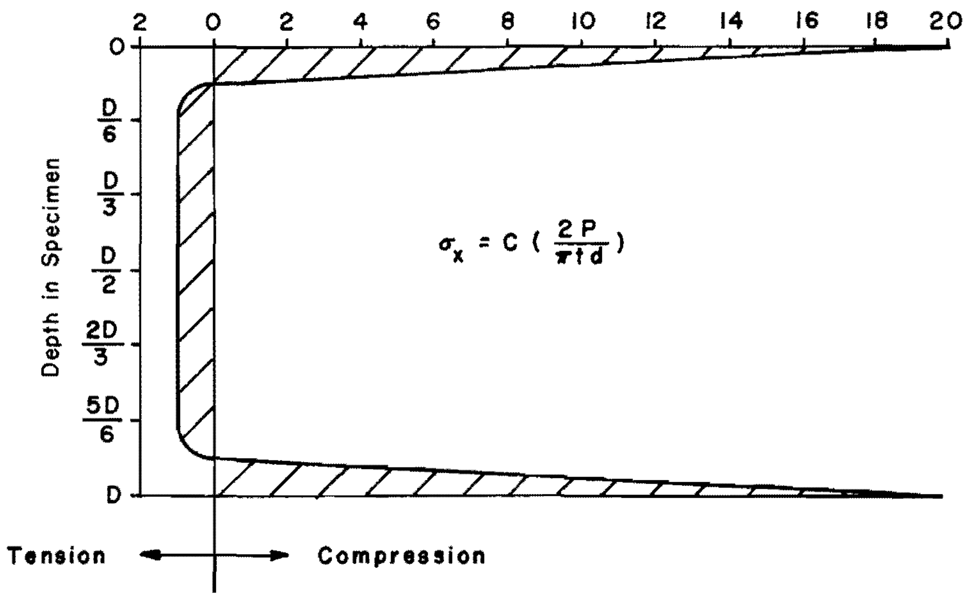


Fig 18. Horizontal stress distributions on the y-axis for loading strip width equal to d/12 (Ref 30).

Deviation from Hooke's Law

It is assumed in the theoretical considerations of the test that strain is proportional to stress. This does not hold in the case of concrete, asphaltic concrete, and stabilized materials. Probably the worst case occurs with bituminous materials. In all of these materials the modulus of elasticity or deformation tends to decrease with increased stress.

Both Wright (Ref 31) and Mitchell (Ref 13) state that a nonlinear stress-strain relationship tends to relieve the more highly stressed parts of the specimen. This would tend to increase the load required to cause failure in the specimen and to give higher strength values. Nevertheless, there is no apparent reason to question seriously the results obtained from indirect tensile testing of nonlinear stress-strain materials provided the specimen fails in tension.

It is also reasonable to conclude that the test is more applicable to brittle materials and that some consideration and test evaluation would be desirable for materials such as asphaltic concrete and bituminous stabilized materials before the test can confidently be used for the evaluation of these materials.

MODES OF FAILURE

It has previously been noted that a basic requirement of the test is that the specimen fail in tension. Several modes of failure have been observed and it is important to distinguish between them.

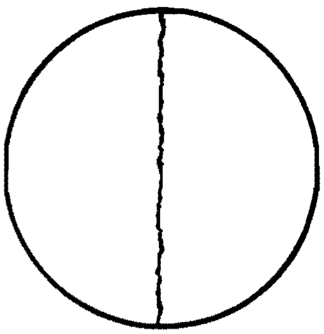
Compression failures might be expected to occur immediately beneath the loads and would appear as localized crushing. This crushing is generally not serious and serves only to increase the area over which the load is applied. Ultimate failure still may occur in either tension or shear.

Rudnick et al (Ref 32) found that the maximum shear stresses occur beneath the surface with the exact location and magnitude of these stresses depending upon the distribution of the applied loads. As shown in the theoretical development for the test, there are no shear stresses acting on the vertical and horizontal diameters. Thus, the vertical and horizontal stresses are principal stresses with the maximum shear stresses acting on a plane at 45 degrees to the vertical. It could, therefore, be expected that a shear failure would intersect the loaded diameter.

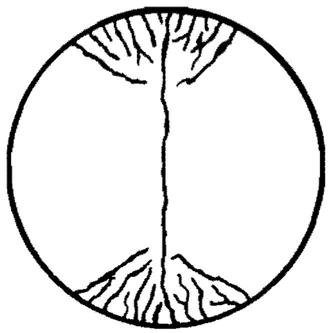
The desired tension failure is caused by the tensile stress acting perpendicular to the loaded diameter. A number of acceptable tensile failure patterns are shown in Fig 19. Mitchell (Ref 13) attributed most of these variations to the characteristics of the loading strip.

Localized crushing with ultimate failure in tension is illustrated in Fig 19b. Mitchell observed this type of failure when no loading strip or a very narrow loading strip was used for testing concrete. The double cleft failure (Fig 19c) was caused by the use of a very large plate which resulted in shearing stresses. Nevertheless, the specimen ultimately still failed in tension. The single cleft failure (Fig 19d) observed by Mitchell always occurred on the bottom side of the cylinder which was the side loaded with the moving head of the machine. All failures of this type occurred with narrow plates, no plates, or rigid plates of masonite.

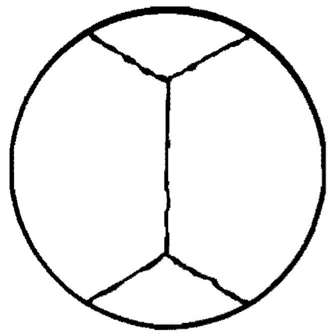
The triple cleft failure (Fig 19e) was first observed by Mitchell while testing Keene's cement cylinders. This type of failure was ascribed to shearing stresses. Rudnick et al (Ref 32), however, analyzed this type of failure and concluded that fracture was initiated along the loaded diameter. The load causing this failure was the highest obtained during the test, and the outer fractures occurred subsequent to the central fracture. Thus, it was concluded that this type of failure can be used to determine tensile strength.



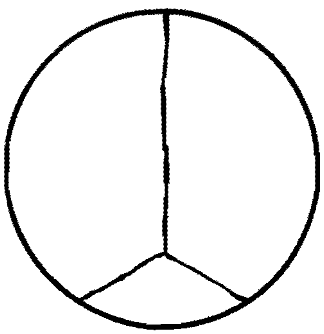
(a) Ideal failure.



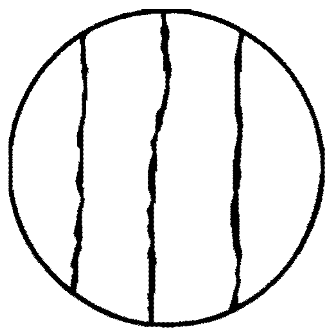
(b) Localized crushing failure.



(c) Double cleft failure.



(d) Single cleft failure.



(e) Triple cleft failure.

Fig 19. Previously observed tensile failures (Refs 13 and 32). Causes: (a) ideal strip, (b) no strip, narrow strip, (c) very wide strip, (d) no strip, narrow strip, rigid strip - occurs on the side of the moving platen.

CHAPTER 4. FACTORS AFFECTING THE INDIRECT TENSILE TEST

Some of the characteristics of the indirect tensile test and the materials tested which may affect the test results are:

(1) load-deformation characteristics of the material tested,
(2) size and dimensions of the specimen,
(3) composition and dimensions of the loading strip,
(4) rate of loading, and
(5) testing temperature.

Characteristics and properties of the test material are not considered in the theoretical development of the test, except in the form of a tensile strength which limits the ultimate applied load. Various investigators (Refs 33 and 34) have noted that such parameters as the modulus of elasticity and Poisson's ratio do influence the specimen during testing. Bawa (Ref 34) concluded that it was inadvisable to use the simple formula $S_T = \frac{2P_{max}}{\pi td}$ for calculating the real tensile strength since the multiaxial state of stress requires consideration of Poisson's ratio and since there is considerable evidence that the failure of concrete depends on strain as well as stress. To date little work has been done toward evaluating the effect of such parameters as the modulus of elasticity and Poisson's ratio. Although some error is undoubtedly introduced into the results, the effect is apparently small enough to allow the test to be applied to a variety of materials, and there does not appear to be any evidence to indicate that the simple formula shown above introduces a significant error. An indirect method of considering the characteristics of the tested material is to evaluate possible factors affecting the test by testing different types of material.

Unfortunately, most of the work involving the indirect tensile test has been concerned with the determination of the tensile strength of concrete and mortar. Exceptions to this are the work conducted by Thompson (Ref 21) on lime-soil mixtures and the work conducted by Messina (Ref 12), Breen and Stephens (Refs 22 and 23), and Livneh and Shklarsky (Ref 24) on asphaltic concrete. Previous findings concerning the factors listed above are presented and summarized below

SIZE AND DIMENSIONS OF THE SPECIMEN

Rudnick et al (Ref 32) discussed size effects in terms of statistical concepts. They postulated that in brittle materials flaws exist which are randomly distributed throughout the volume and that fracture occurs when the applied stress reaches a critical value at a flaw which is properly positioned to initiate a crack. Thus it might be expected that larger specimens would exhibit lower strengths and more uniform results since a larger amount of material is being stressed. Experimental studies (Refs 19, 20, and 31) have substantiated this by showing that an increase in the overall size of concrete specimens causes a reduction in the average tensile strength and a reduction in dispersion of the individual test results.

Another aspect of the test concerning the dimensions of the specimen is the ratio of the length to the diameter. According to the theory of the test the length-diameter ratio of the specimen should have no effect since the theory is independent of thickness. Studies by Grieb and Werner (Ref 16) and Messina (Ref 12), conducted to substantiate this fact, have generally shown no effect from changes in the length-diameter ratio. Rudnick et al (Ref 32) investigated photoelastically the stress distributions at the ends of both long cylinders and thin disks. These specimens were 1 inch in diameter and either 1-inch or 1/8-inch long. The experiments indicated no detectable difference between stresses developed in the end of a 1-inch-thick cylinder and the stresses developed in a 1/8-inch-thick disk at comparable applied loads. Thaulow (Ref 17) adds additional support by reporting that tests made in Denmark (Ref 35) and Norway (Ref 36) indicated that the indirect tensile strength is largely independent of the length and diameter of the specimen.

From the above findings it was concluded that the dimensions of the specimen produce no significant effects. It is apparent, however, that large specimens provide more uniform test results, and thus are more desirable.

COMPOSITION AND DIMENSIONS OF THE LOADING STRIP

The indirect tensile test is based on the state of stress which develops from an idealized line load. In reality such a loading condition cannot occur. In addition, it is probably beneficial to apply a distributed load since it (1) reduces the magnitude of the maximum compressive and shear stresses and (2) causes the stresses acting perpendicular to the loaded diameter to change from tension to compression at the edges, thus minimizing the effect of surface irregularities in the specimen. In general, the purposes of applying the load through some type of loading strip are (1) to distribute the load uniformly over an appreciable width and (2) to distribute the load by reducing the effects of irregularities in the surface of the test specimen.

Although it is apparently advantageous to apply load through a loading strip, studies have indicated that the type of material, shape, and dimensions of the loading strip may have a definite effect on the stress distribution, type of failure, and test results. It is also possible that the desirable characteristics of the loading strip will vary with the type of material being investigated.

Characteristics and properties of the loading strip have received considerable attention. Mitchell (Ref 13) conducted tests on high-strength concrete cylinders with strain gages mounted on their faces. In the first test, with cardboard load strips, the strain increased constantly up to the failure load. In the second test, using masonite strips, the strain increased constantly and, for the same load, the strains were similar to the first test. At the failure strain of the first test, however, there was a strain reversal in the specimen tested with masonite and the specimen failed from the bottom. Final failure was similar to that observed with narrow strips and resembled a single-cleft failure (Fig 19c). The author concluded that masonite strips do not provide good bearing over the entire width of the strip. Wright (Ref 31) conducted tests on concrete cylinders using wood, steel, and rubber loading strips. He found that the strength results did not differ significantly for wood and rubber strips of the same size but that steel strips resulted in lower and less uniform results. In this case he adopted plywood for use as a loading strip since it was easier to use than rubber. Grieb and Werner (Ref 16) conducted a limited series of tests on concrete using plywood and neat Lumnite cement loading strips. No appreciable differences in strength were noted for these two materials. Addinall and Hackett (Ref 37) studied the effect of platen conditions on the strength results for high-strength autoclaved plaster using platen materials ranging from steel to hard rubber. Unlike other studies it was concluded that softer loading strips produce higher strengths and higher dispersion values. The lowest value of dispersion was found to occur when no loading strip was used with the load being applied through the steel platens. No explanation of the difference between these results and Wright's results was offered. It was also found photoelastically that this system of loading produces a stress distribution closely resembling the theoretical distribution for line contact. Simon and Aust (Ref 38) and Ramesh and Chopra (Ref 33) used wooden strips for extensive testing while Igbal Ali et al (Ref 39) used rawhide loading strips. Messina (Ref 12) in the indirect tensile test evaluation of asphaltic concrete used aluminum loading strips with concave faces.

Considerable attention has also been devoted to the determination of the best loading-strip dimensions. Wright (Ref 31) conducted a limited study concerning the effect of strip width and thickness on the indirect tensile results for concrete. It was found that varying the width of plywood strips (1/2 x 1/8 inch; 1 x 1/8 inch) did not have a significant effect on the observed results. Increasing the thickness from 1/8 inch to 1/4 inch reduced the observed strength. His calculations indicated that it was unlikely this effect was due to random error, but no reasons for the observed behavior were suggested. Mitchell (Ref 13) conducted a study on high-strength concrete using cardboard strips ranging in width from 3/4 inch to 2 inches. The test results indicated that the strip width did not seriously affect the strength of the specimens but did affect the rupture characteristics. He found that wide strips usually resulted in cleaner breaks. Narrow strips resulted in shattering, while wide strips caused double-cleft failures with large pieces which, in some cases, had not split completely to the central fracture.

Although the findings cited above do not show conclusive evidence of the best type of material and dimensions for loading strips, ASTM has adopted a tentative standard for the determination of the indirect tensile strength of concrete (Ref 40). This standard recommends as a loading strip the use of nominal 1/8-inch-thick plywood with a width of approximately one inch.

From an evaluation of these works it would appear that there is no definitely accepted knowledge concerning the desirable composition and width of the loading strip. Particularly, there is little information resulting from the testing of stabilized materials and no information involving deformation measurements. Therefore, additional study concerning the desirable characteristics and properties of loading strips seems desirable, especially for stabilized materials.

RATE OF LOADING AND TESTING TEMPERATURE

Information concerning the effect of testing temperature and loading rate on indirect tensile test parameters is limited. Mitchell (Ref 13) conducted tests on high-strength concrete and reported that increased speed of testing resulted in higher observed indirect-tensile strengths.

Messina (Ref 12) conducted a limited study of the effect of testing temperature on the indirect tensile strength of asphaltic concrete. This investigation showed that strength was increased by a factor of about 2.5 for a decrease of testing temperature from 77°F to 50°F . Breen and Stephens (Refs 22 and 23) conducted a more extensive study of the effects of temperature on indirect tensile test parameters. Based on their tests it was concluded that as the temperature decreases asphaltic concrete becomes more brittle and the load at fracture increases slowly. For a decrease in temperature from 40°F to 0°F the tensile strength increased by 20 to 25 percent. It was also concluded that both the ultimate deflection and the work required to fracture the specimen decrease with a decrease in temperature.

More important than the change in strength associated with increased loading rates and decreased temperature is the change in the character of the stress-strain relationship exhibited by the material being tested. Both a decreased testing temperature and an increased loading rate tend to produce more brittle behavior and a more linear stress-strain relationship which is advantageous according to test theory.

This page replaces an intentionally blank page in the original.
-- CTR Library Digitization Team

CHAPTER 5. EXPERIMENTAL EVALUATION AND DEVELOPMENT OF THE INDIRECT TENSILE TEST

On the basis of previous work the indirect tensile test seems to be the best test currently available for determining the tensile properties of stabilized materials. The test has many practical advantages such as

(1) The coefficient of variation of the test results is low.

(2) Failure is not seriously affected by surface irregularities in the specimen.

(3) Failure is initiated in a region of relatively uniform tensile stress.

Its major disadvantages are the complexity of the theory and the fact that the loading conditions do not resemble field loading conditions.

Previous evaluation, both theoretical and experimental, has established the influence of certain factors which can affect test results. It has been shown that the length-to-diameter ratio of the specimen tested has little effect on the resulting strength parameter, and it has been shown that an increase in the overall specimen size results in more uniform strength data, but slightly reduced average strength values.

It has also been established that the composition and dimensions of the loading strips affect strength results and type of failure. However, previous tests do not indicate the best type of material and dimensions of the loading strips. In addition, there is little information on the effects of testing temperature and loading rate.

Unfortunately, most of the experimental evaluation of the indirect tensile test has been conducted on concrete and has not included deformation measurements. This fact, along with the lack of conclusive evidence concerning the most desirable composition and width of the loading strips and the lack of temperature and loading rate information, makes it important to evaluate the indirect tensile test using materials other than concrete and to include deformation measurements. The findings of such an evaluation along with previously reported findings will aid in establishing standard test procedures for future studies.

The objectives of this initial phase of investigation were to develop equipment and a technique for conducting the indirect tensile test and, as a part of this development, to evaluate the effect of (1) composition of loading strip, (2) width of loading strip, (3) testing temperature, and (4) loading rate on several test parameters including the indirect tensile strength, vertical failure deformation, and a load-vertical deformation modulus.

EQUIPMENT

Equipment and facilities used in the evaluation of factors affecting the indirect tensile test were developed during the process of testing. A total of six additional series of tests were conducted for the purpose of evaluating the effect of such factors as composition and width of loading strip. During these experimental programs many difficulties and problems occurred. In order to eliminate these difficulties or to reduce or control the magnitude of their effect, it was necessary to modify the existing equipment and facilities or, in some cases, to develop new facilities. Because of these difficulties and the changes in equipment and measuring techniques, the data from these various experimental programs have not been included in this report.

The basic testing equipment is shown in Fig 20 and consists of an adjustable loading frame, a closed-loop electrohydraulic loading system, and a loading head. The loading device is a modified, commercially available shoedie with upper and lower platens constrained to remain parallel during tests. Other loading heads were considered, including one which allowed the upper platen to rotate about an axis perpendicular to the longitudinal axis of the specimen. This would allow the platens to be in nominal contact along the length of a specimen which is not of uniform diameter. However, the specimen would have to have a constant change of diameter per unit length of specimen in order for uniform contact to be achieved, and specimens could fail at one end if there were a strength differential in the specimen. Such a device would be undesirable for routine testing, and it was felt that non-rotating platens would provide a better measure of the average strength of the specimen. Thus, a loading device with rigid parallel platens (Fig 21) was chosen.

Another piece of equipment, a device for measuring the transverse strain in a specimen, has also been developed for use. This apparatus was needed to obtain a measure of specimen deformation in the direction of the tensile stresses causing failure. These deformations along with the tensile stresses

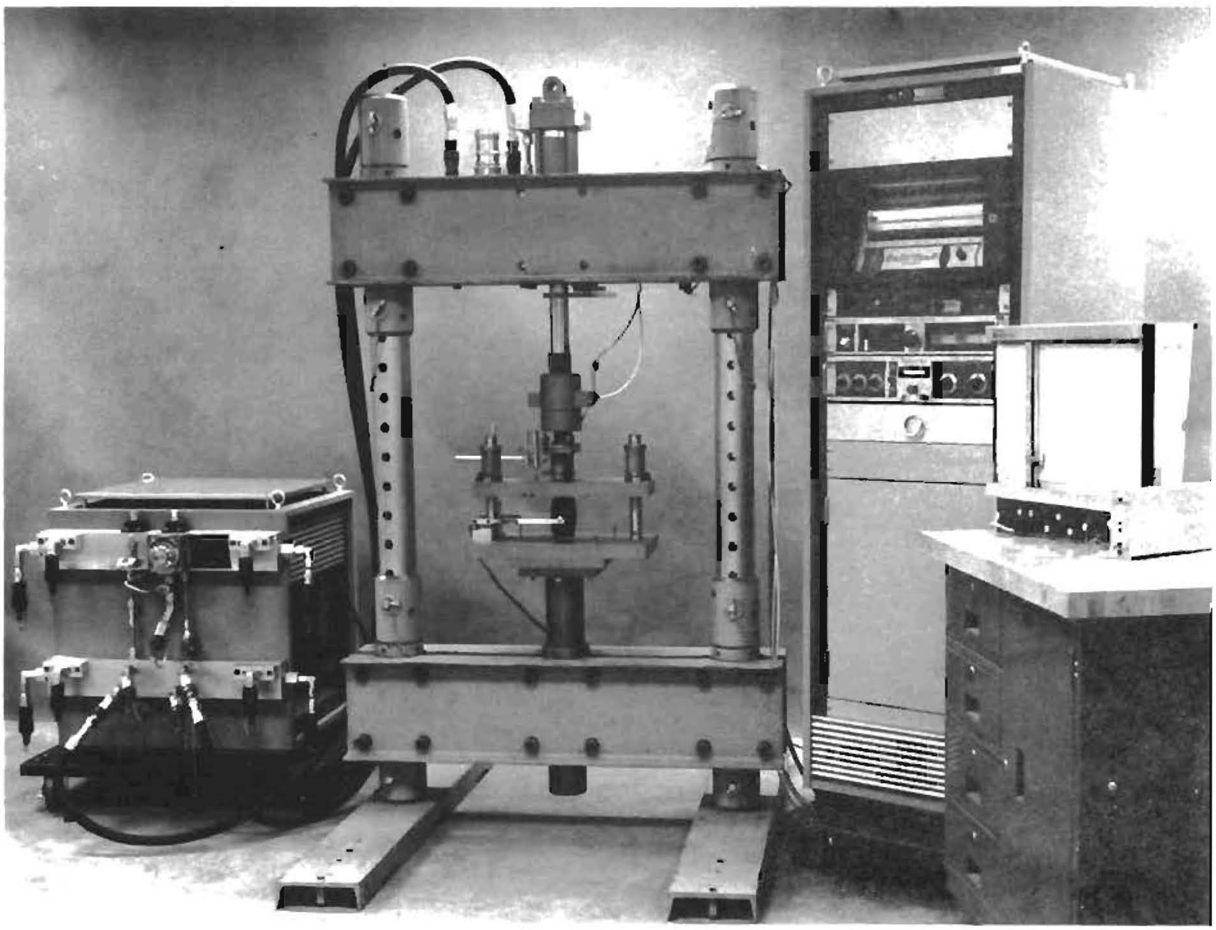


Fig 20. Basic indirect tensile testing equipment.

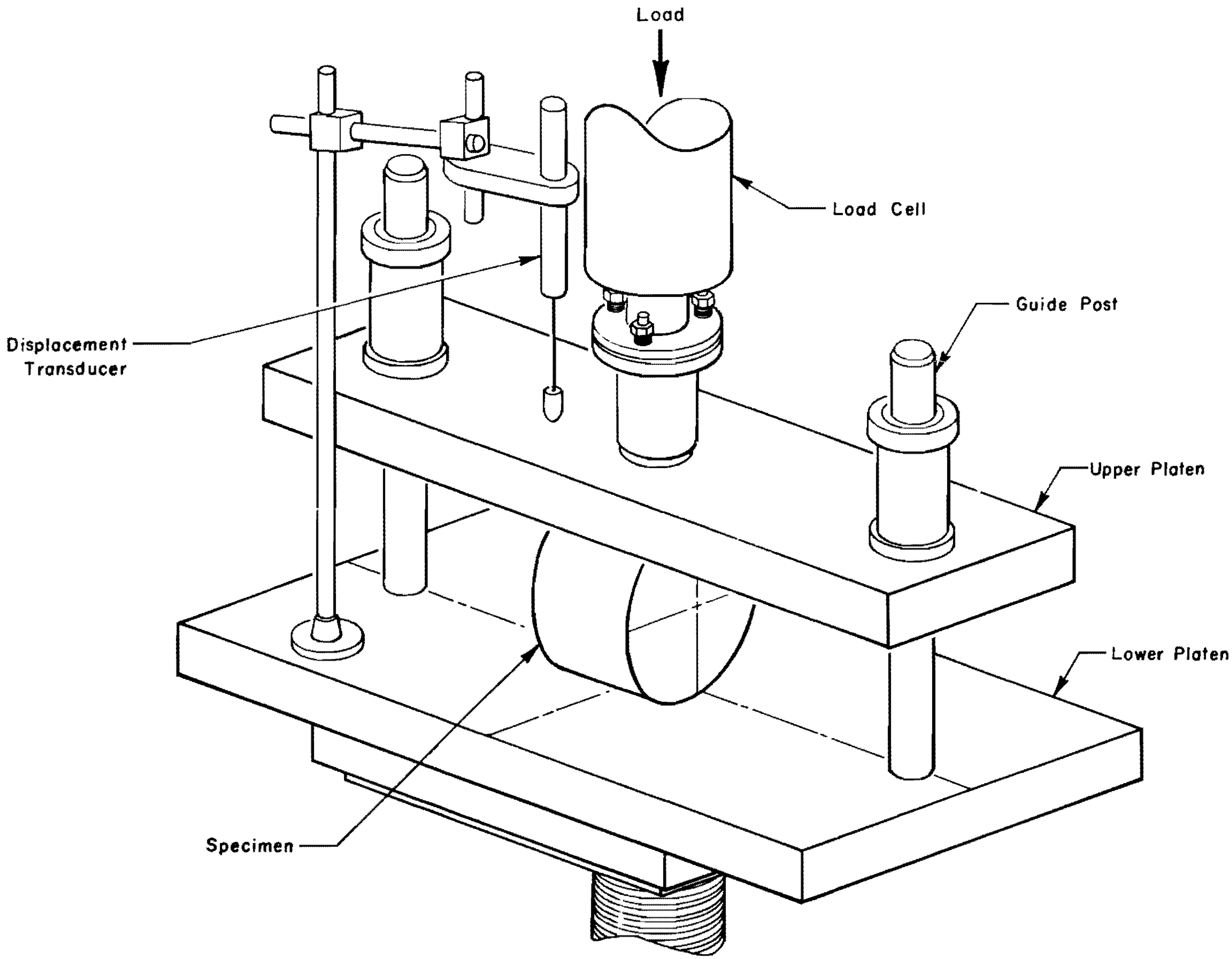


Fig 21. Loading head with rigid parallel platens.

can be used to obtain an estimate of the indirect tensile modulus of deformation. The measuring device consists of two cantilevered arms with attached strain gages and is shown in Fig 22. Movements or deflections of the arms at the points of contact with the specimen have been calibrated with the output from the strain gages. Vertical deformations are measured by a DC linear-variable-differential transformer which also is used to control the rate of load application by providing an electrical signal related to the relative movements of the upper and lower platens. All measurements are recorded on two X-Y plotters.

EXPERIMENTAL PROGRAM

The primary objective of the experimental program was to evaluate the effects produced by the composition of the loading strip, width of loading strip, testing temperature, and loading rate. After evaluation of test findings a decision was made for tentatively standardizing the indirect tensile test for future testing. The primary statistical parameters for the evaluation were the standard deviation and the coefficient of variation used as measures of dispersion; however, mean values are included for comparison.

Three test series were conducted which included samples of asphaltic concrete and cement-treat gravel. The asphaltic concrete consisted of crushed limestone and 5.3 percent AC-10; the cement-treated gravel was a rounded gravel obtained near Seguin, Texas, treated with 6 percent type I portland cement. All specimens were 4.0 inches in diameter with a nominal height of 2.0 inches and were compacted using the Texas automatic gyratory shear compactor. Details concerned with the mix design, sample preparation, and curing of the asphaltic concrete and cement-treated gravel are included in Appendices A and B, respectively.

In these preliminary tests the following parameters were defined and evaluated:

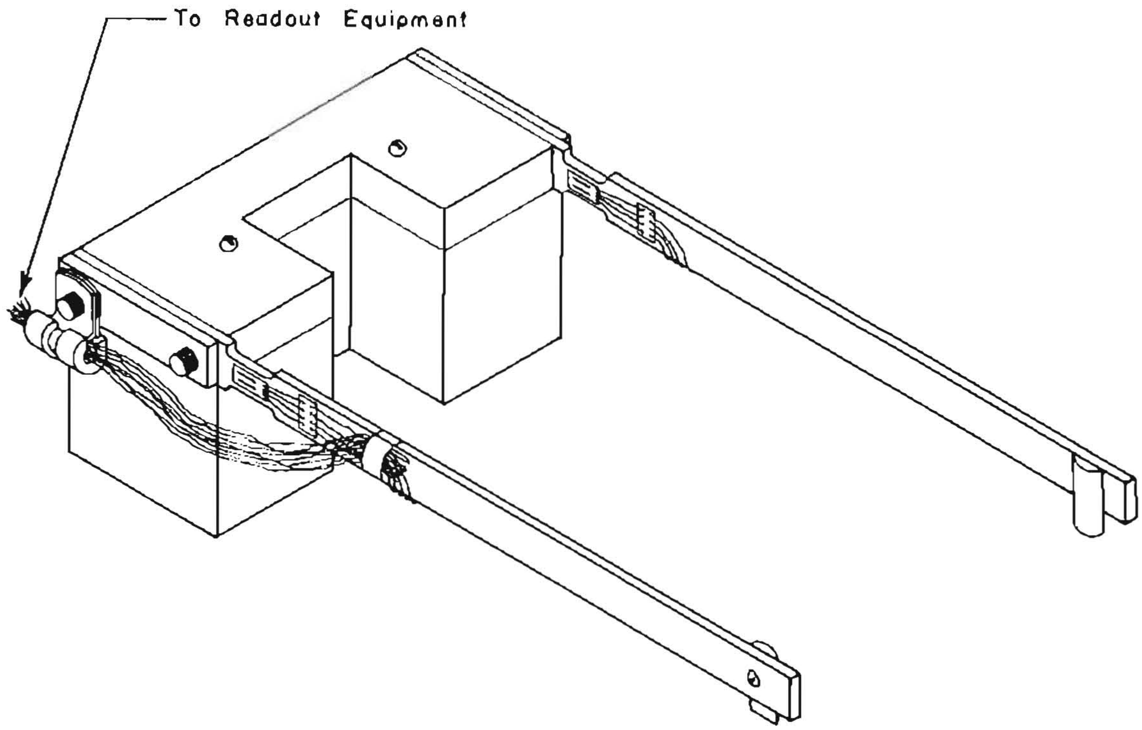
(1) Indirect Tensile Strength, $S_T = \dfrac{2P_{max}}{\pi t d}$

where P_{max} = maximum total load, lbs;

t = average height of specimen, inches;

d = nominal diameter of specimen, inches.

(2) Vertical Failure Deformation - Vertical deformation of the specimen in inches at maximum load including the deformation in the loading



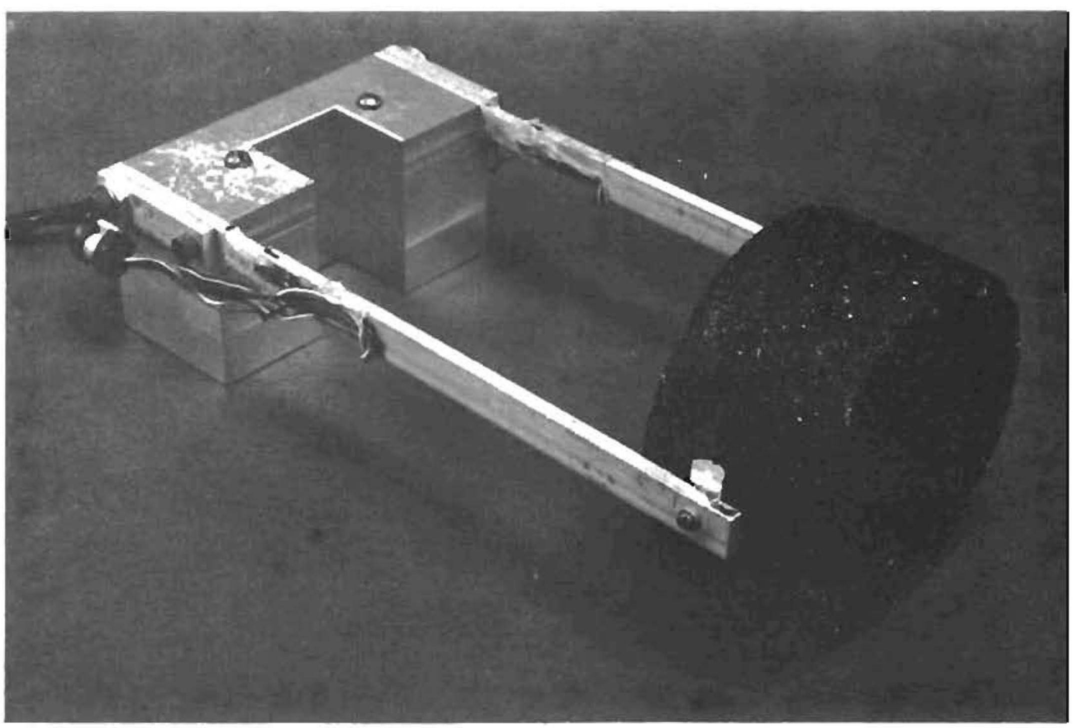


Fig 22. Lateral-strain measuring device.

strip*. This deformation was assumed to be equal to the movement of the upper platen from the point of initial load application to the point of maximum load as measured by the DCDT and recorded on the load-vertical deformation plot.

(3) Tangent Modulus of Vertical Deformation - Slope of the load-vertical deformation relationship prior to failure as defined by a regression analysis. Approximately ten points between the points of initial load and maximum load were obtained from the load-vertical deformation relationship and analyzed by the method of least squares to obtain the best estimate of the slope of a straight line through the points. Corrections were made for deformations in the neoprene strips prior to the regression analysis.

Evaluation of Composition and Width of Loading Strip

The first phase of testing was concerned with the evaluation of the type of material used for the loading strip and the width of the loading strip. Initially, plywood loading strips were considered and were used in testing because of previous recommendations. These previous studies, however, did not involve deformation measurements. Since the measured vertical deformation included the deformation of the loading strip and since plywood strips deform appreciably, it was necessary to subtract the loading-strip deformation from the measured deformation in order to obtain an estimate of the vertical deformation of the specimen. Such corrections were difficult and probably erroneous due to the fact that (1) wood is heterogeneous and variable, (2) wood deforms appreciably at higher stresses, and (3) wood does not exhibit a linear stress-strain relationship. For these reasons wood was discarded as a possible loading-strip material.

Other strip materials investigated were stainless steel and neoprene. These two materials were chosen because they were readily available, easily specified, and represent, to a certain degree, extremes with regard to rigidity. Strip widths of 0.5 inch and 1.0 inch were used. An additional variable involved the application of load directly through the platens with no loading

* Corrections were made for deformations occuring in the neoprene load strip in some parts of the analysis.

strips. All specimens were tested at $75^{o}F$ at a loading rate of 0.5 inch per minute. This phase of the testing was divided into two parts. The first part involved the testing of asphaltic concrete which was considered a questionable material since it exhibits plastic characteristics rather than purely elastic characteristics as assumed by theory and because there was lack of information concerning the use of the indirect tensile test for testing asphaltic materials. The second part of the testing involved cement-treated gravel, a more brittle material, which more closely approximates the behavior of an elastic material.

The experimental designs for these two series of tests are shown in Figs 23a and 23b. Both were full-factorial, randomized designs involving two types of strips and two strip widths. Analyses of variance of the log-variances were conducted for these variables. No statistical analysis was conducted for the variable involving the direct application of load with no loading strip, although subjective comparisons were made.

Discussion of Findings Using Asphaltic Concrete. The initial test series in the evaluation of the effect of composition and width of loading strip was conducted on asphaltic concrete specimens. The evaluation included the test parameters of indirect tensile strength, vertical failure deformation, and tangent modulus of vertical deformation. The data are summarized in Table 2 and the results of the analysis of variance of the log-variance are summarized in Table 3.

The findings concerning indirect tensile strength are shown in Fig 24 where it can be seen that steel loading strips and 0.5-inch strip widths result in higher standard deviations of the data. The 1-inch neoprene strip gave the minimum standard deviation and coefficient of variation (Table 2) and the platen condition resulted in the maximum. The analysis of variance of the log-variances indicated that the lower standard deviation associated with the 1.0-inch widths was significant ($\alpha = 0.05$)*. The reduced dispersion associated with the 1-inch width, however, was not significant.

---

* Alpha (α) represents the probability that in reality there is no significant difference between the parameters which were statistically compared even though the analysis indicates that there is a difference. An alpha of 0.05 or 5.0 percent indicates in general that not more than 5 times out of 100 could the observed difference have occurred purely by chance for samples of the same population.

WIDTH OF LOADING STRIP

TYPE OF LOADING STRIP

	0.5"	1.0"	Platens
Neoprene	(8)	(8)	
Stainless Steel	(8)	(8)	(8)

a. Design for evaluating the effect of strip type and width for asphaltic concrete.*

WIDTH OF LOADING STRIP

TYPE OF LOADING STRIP

	0.5"	1.0"	Platens
Neoprene	(5)	(5)	
Stainless Steel	(5)	(5)	(5)

b. Design for evaluating the effect of strip type and width for cement-treated gravel.*

LOADING RATE - in/min

TESTING TEMPERATURE ° F

	0.05	0.14	0.50	2.0	4.0	6.0
-10	(1)	(1)	(1)	(1)	(1)	(1)
20	(1)	(1)	(1)	(1)	(1)	(1)
50	(1)	(1)	(1)	(1)	(1)	(1)
80	(1)	(1)	(1)	(1)	(1)	(1)
110	(1)	(1)	(1)	(1)	(1)	(1)
140	(1)	(1)	(1)	(1)	(1)	(1)

Note: 3 replications were conducted.

c. Design for evaluating the effect of testing temperature and loading rate for asphaltic concrete.*

* Numbers in parentheses indicate the number of observations.

Fig 23. Experiment designs.

TABLE 2. SUMMARY OF THE DATA FROM THE EVALUATION OF THE EFFECT OF STRIP TYPE AND WIDTH FOR ASPHALTIC CONCRETE

Type of Loading Strip		Neoprene				Stainless Steel		Platens (No Strips)
Strip Width, inches		0.5		1.0		0.5	1.0	∞
No. Specimens		8		8		8	8	8
Indirect Tensile Strength	Average, psi	105		108		106	103	111
	Std. Dev., psi	7.0		2.0		8.1	4.2	9.8
	Coef. of Var., %	6.7		1.9		7.6	4.1	8.8
Vertical Failure Deformation*		Strip Correction	No Correction	Strip Correction	No Correction			
	Average, inches	.0815	.0992	.0769	.0965	.0603	.0565	.0572
	Std. Dev., inches	.00623	.00640	.00466	.00490	.00708	.00333	.00609
	Coef. of Var., %	7.6	6.4	6.1	5.1	11.70	5.9	10.6
Tangent Modulus of Vertical Deformation	Average, lb/in	17,620		19,470		21,010	23,070	23,070
	Std. Dev., lb/in	1571		1423		2470	1845	2691
	Coef. of Var., %	8.9		7.3		11.8	8.0	11.7

*Two analyses were conducted for vertical failure deformation.

(1) Strip Correction. Deformation of the neoprene loading strip was estimated and subtracted from the total measured deformation to obtain specimen deformation. No correction was made for the deformation of the stainless steel strip.

(2) No Correction. No corrections were made for either the neoprene or stainless steel strips.

TABLE 3. SUMMARY OF THE ANALYSIS OF VARIANCE OF THE LOG-VARIANCE OF THE EFFECT OF STRIP TYPE AND WIDTH FOR ASPHALTIC CONCRETE

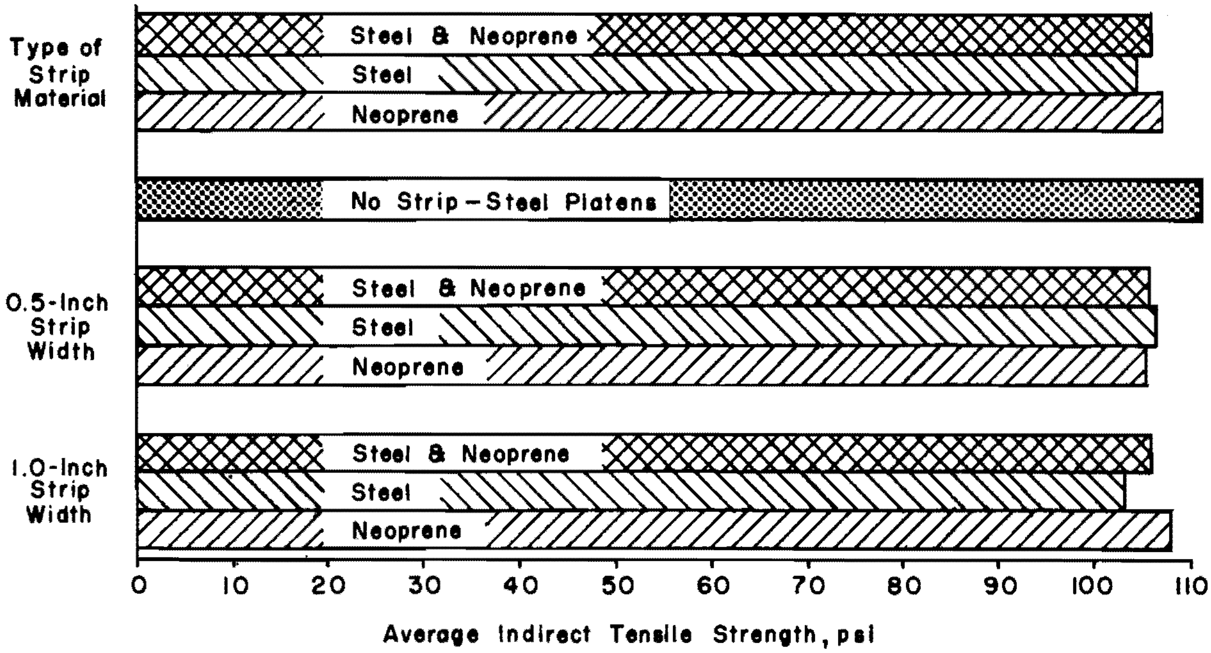
	Source of Variation	Degree of Freedom		Mean Square		F		Significance Level[a], %	
Indirect Tensile Strength psi	Strip Type	1		0.419		4.46		None	
	Strip Width	1		1.549		16.50		5	
	Interaction	1		0.276		2.94		None	
	Error	4		0.094					
Vertical Failure Deformation, in*		Strip correction	No correction	Strip correction	No correction	Strip correction	No correction	Strip correction	No correction
	Strip Type	1	1	0.0973	0.0673	1.08	1.14	None	None
	Strip Width	1	1	0.1658	0.2901	1.83	4.91	None	None
	Interaction	1	1	0.1857	0.0897	2.05	1.52	None	None
	Error	4	4	0.0904	0.0590				
Tangent Modulus of Vertical Deformation lb per in	Strip Type	1		0.1535		1.33		None	
	Strip Width	1		0.0633		0.55		None	
	Interaction	1		0.0374		0.33		None	
	Error	4		0.1150					

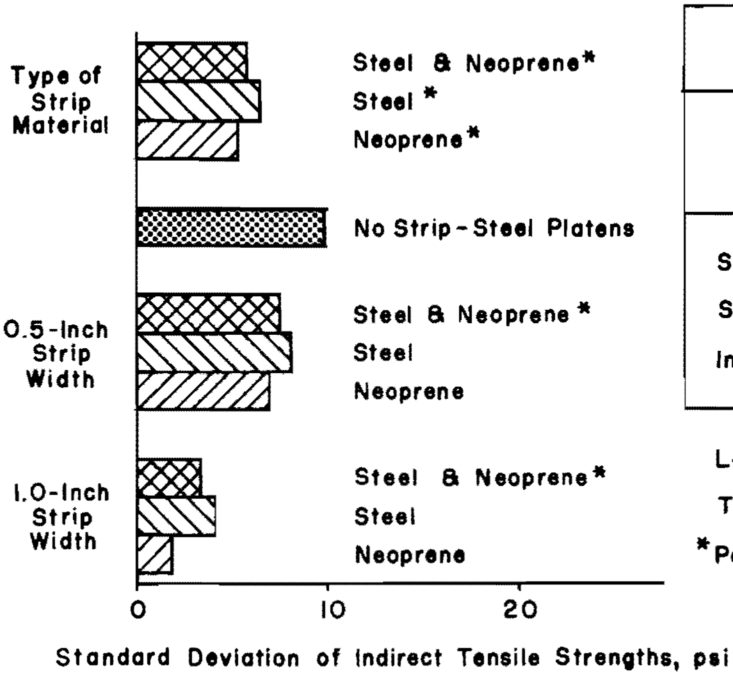
*Two analyses were conducted for vertical failure deformation.

(1) Strip Correction. Deformation of the neoprene loading strip was estimated and subtracted from the total measured deformation to obtain specimen deformation. No correction was made for the deformation of the stainless steel strip.

(2) No Correction. No corrections were made for either the neoprene or stainless steel strips.

[a]If significance level is greater than 10 percent, it is called "none".





Analysis of Variance of Log-Variance

Source of Variation	Significance: Alpha Level	Significance
Strip Type	—	None
Strip Width	5 %	Significant
Interaction	—	None

Loading Rate = 0.5 in/min

Testing Temperature = 75° F

*Pooled Standard Deviation

Fig 24. Effect of type and width of loading strip on the dispersion and average values of indirect tensile strength for asphaltic concrete.

The vertical failure deformations obtained in this series of tests are shown in Figs 25 and 26. The deformations shown for the steel strips in both figures include the deformation occurring in the strips as well as in the specimen*. In Fig 26 the deformations of the neoprene strip were determined experimentally and subtracted from the measured deformation to yield the values shown; however, in Fig 25 no corrections were made for strip deformation.

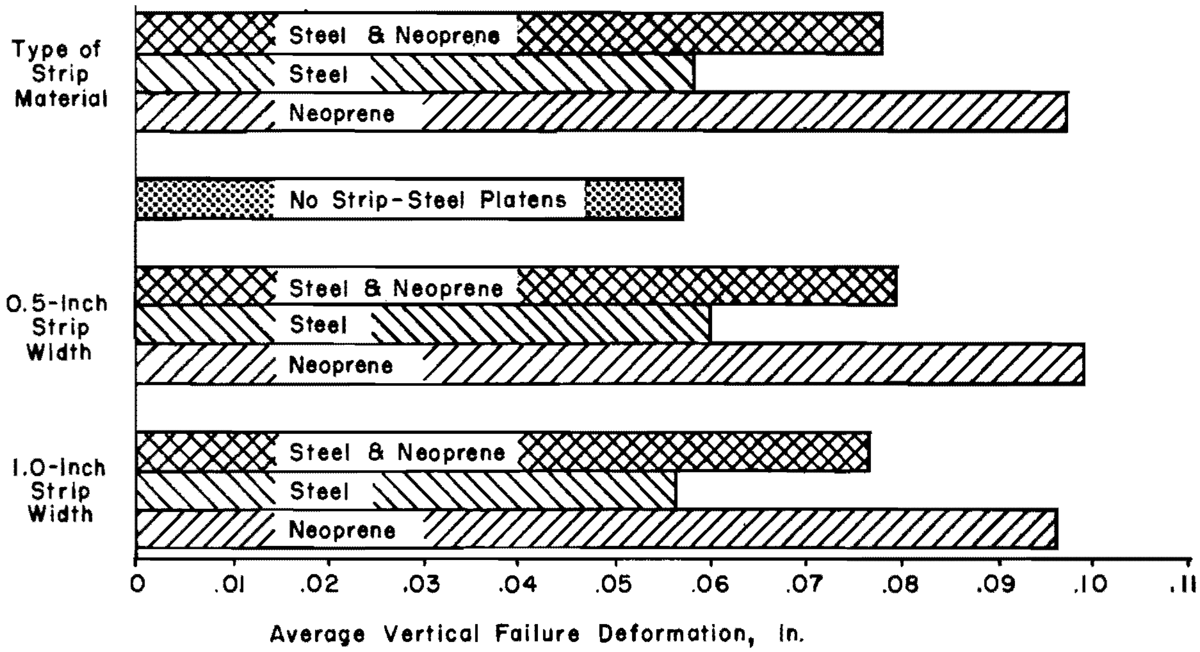
Dispersion values for both the two types of strips were essentially constant; the standard deviations, however, were lower for the 1.0-inch strips than for the 0.5-inch strips. The dispersion associated with the platen loading condition was essentially equal to that of the different strip types. The minimum standard deviation occurred for the 1.0-inch steel strip condition; the minimum coefficient of variation occurred for the 1.0-inch, uncorrected neoprene strip (Table 2). The analysis of variance of the log-variance indicated no significant differences for the two types of loading strip materials; however, the lower dispersion value associated with the 1-inch width strip was significant ($\alpha = 0.10$) for the uncorrected neoprene data (Fig 25), but not for the corrected neoprene data.

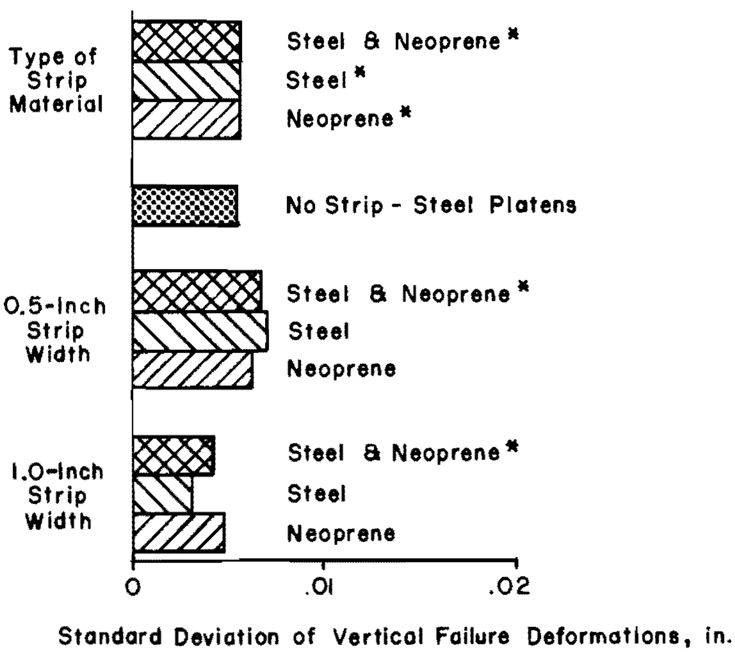
The analysis of the tangent modulus is summarized in Fig 27 and Table 2. The moduli dispersion for specimens tested with steel strips appears substantially higher than for the neoprene strips. As previously observed the 1.0-inch width for both steel and neoprene produced a lower standard deviation than the 0.5-inch width. The data obtained with only the platens had a slightly higher value of dispersion than the data associated with either of the finite widths and substantially higher than for any given test condition. None of these observed differences were statistically significant.

In summary it was found that neoprene strips resulted in higher mean values for strength and failure deformation and lower mean values for the tangent modulus. Likewise, 1.0-inch width strips produced higher mean values for the tangent modulus and slightly higher values for strength but lower mean failure deformations.

---

* The deformation of the steel strip at ultimate load was determined experimentally to be approximately 0.005 inch.





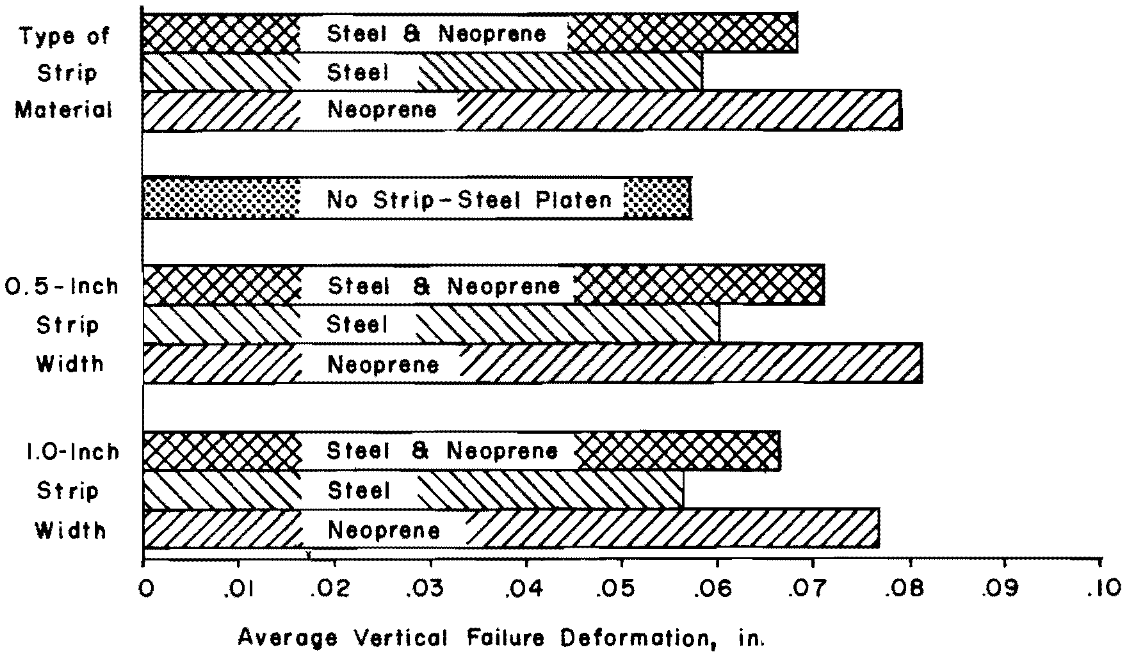
Analysis of Variance of Log-Variance		
Source of Variation	Significance	
	Alpha Level	
Strip Type	—	None
Strip Width	10%	Slightly Significant
Interaction	—	None

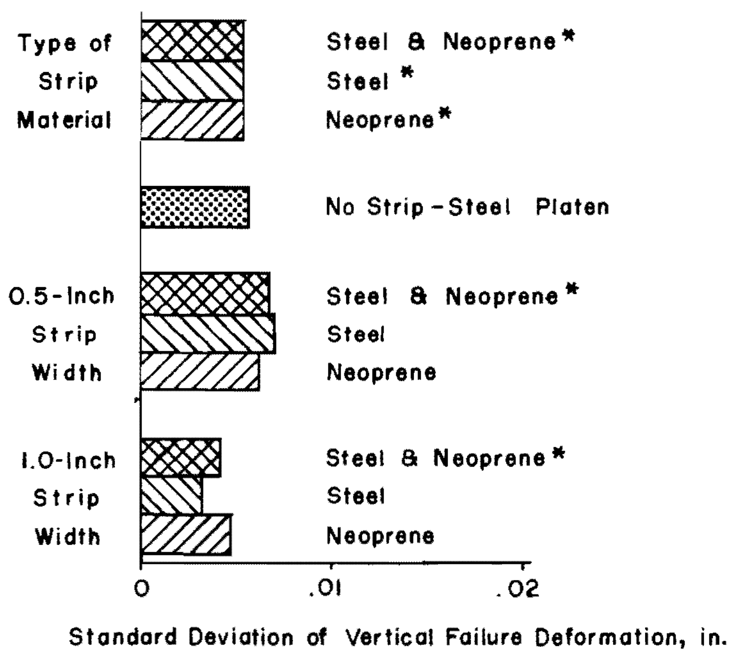
Loading Rate = 0.5 in/min

Testing Temperature = 75 °F

*Pooled Standard Deviation

Fig 25. Effect of type and width of loading strip on the dispersion and average values of vertical failure deformation for asphaltic concrete. (No corrections were made for deformation of loading strip.)





Analysis of Variance of Log-Variance

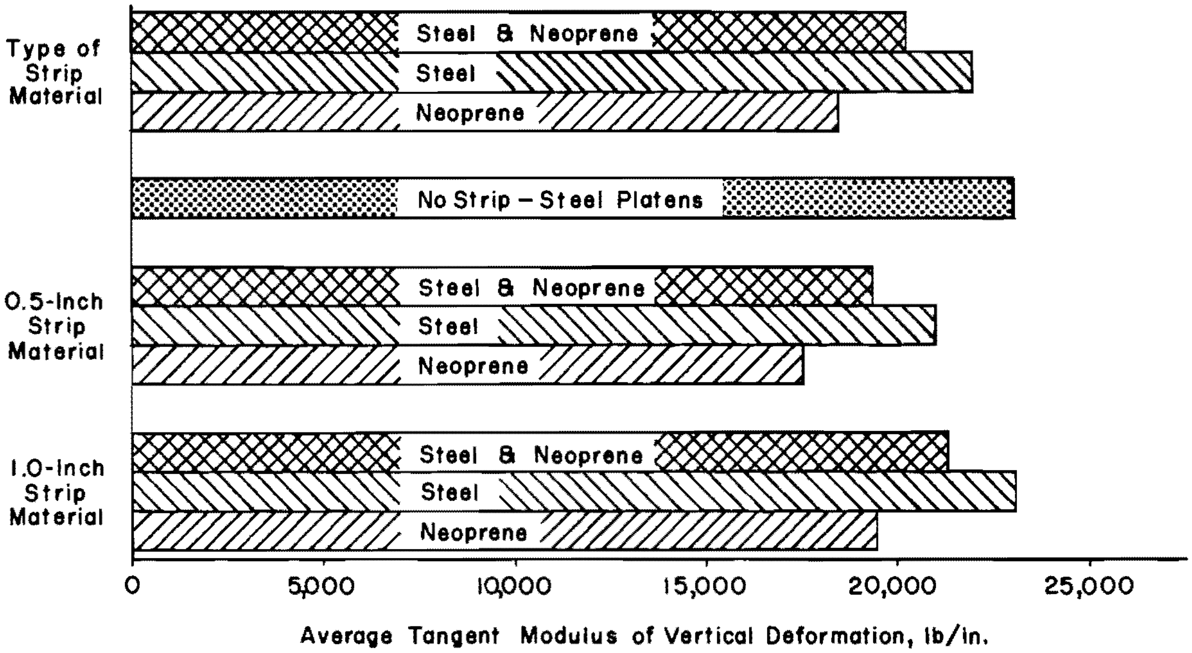
Source of Variation	Significance: Alpha Level	
Strip Type	—	None
Strip Width	—	None
Interaction	—	None

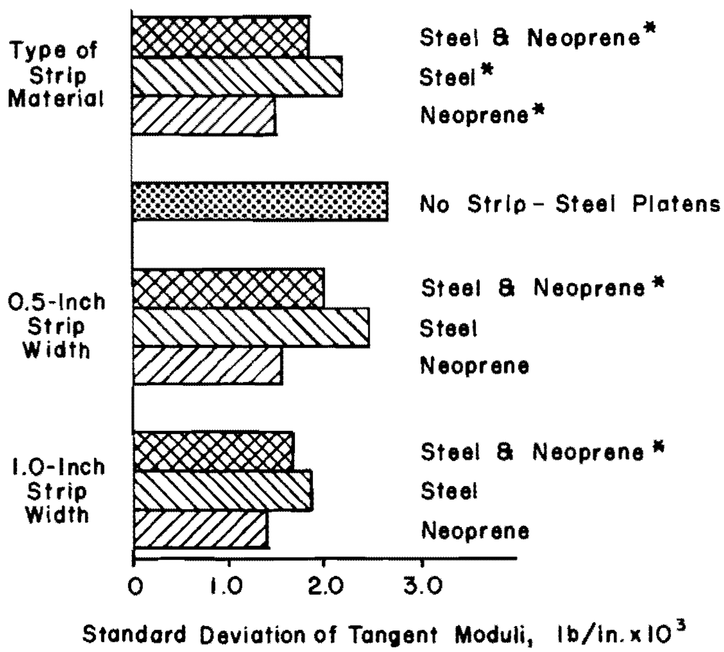
Loading Rate = 0.5 in/min

Testing Temperature = 75°

*Pooled Standard Deviation

Fig 26. Effect of type and width of loading strip on the dispersion and average values of vertical failure deformation for asphaltic concrete. (Corrections were made for deformation of neoprene loading strip.)





Analysis of Variance of Log-Variance		
Source of Variation	Significance	
	Alpha Level	
Strip Type	—	None
Strip Width	—	None
Interaction	—	None

Loading Rate = 0.5 in/min

Testing Temperature = 75°

* Pooled Standard Deviation

Fig 27. Effect of type and width of loading strip on the dispersion and average values of tangent modulus of vertical deformation for asphaltic concrete.

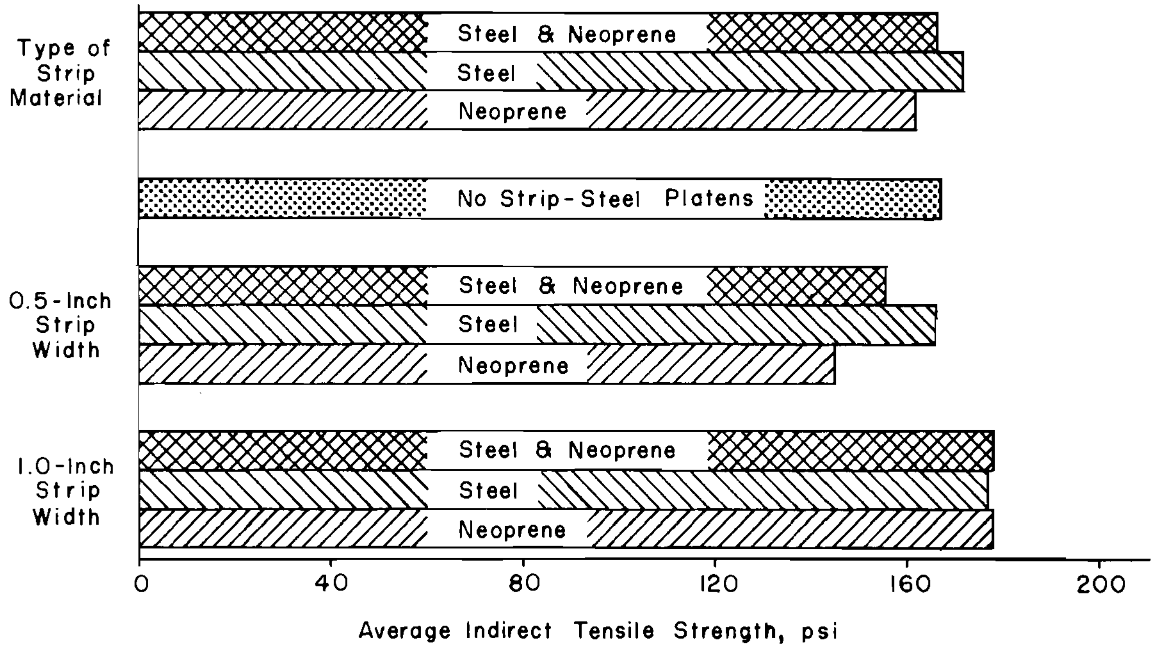
In general it may be noted that the standard deviations of the data obtained from specimens tested with steel strips are slightly higher than those obtained from specimens tested with neoprene; however, the differences are small and are not significant. There is apparently a definite advantage to using the 1-inch width strip because of the reduced dispersion of the data obtained from specimens tested with both steel and neoprene loading strips. This reduced dispersion occurred for all measured parameters and was significant for strength and the uncorrected failure deformations.

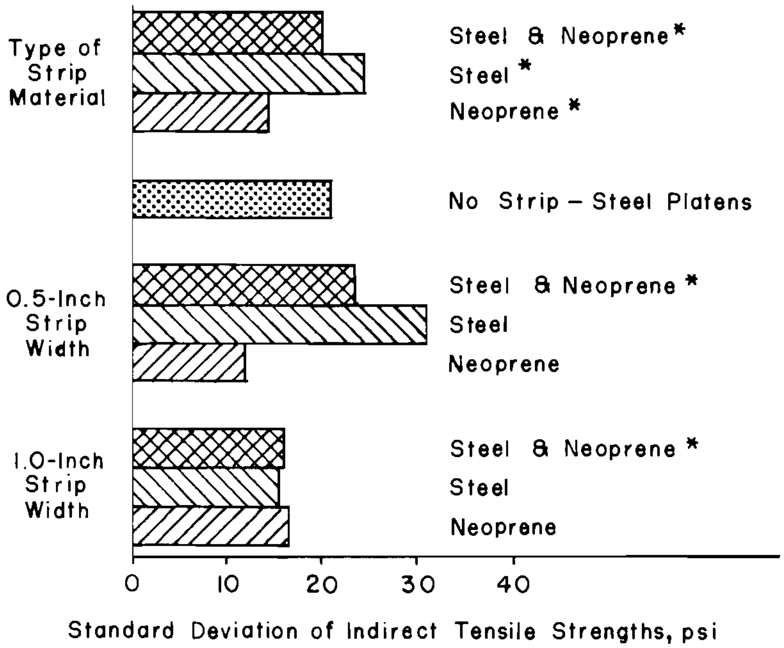
On the basis of this analysis it could be recommended that future testing be conducted using a 1.0-inch width, neoprene loading strip. Nevertheless, in view of the practical advantages of using steel and the small and statistically insignificant differences between the dispersion of the data obtained from the steel and neoprene it is felt that a 1.0-inch steel loading strip is more desirable. Results published with regard to concrete and mortar, however, have generally recommended a softer, more flexible loading strip material. In addition, it has been reported that the width of the loading strip has an effect on the type of failure. On the basis of the above recommendation and the lack of a significant advantage of one material over the other, it was desirable to investigate the effects of both type and width of loading strip on a more brittle material, cement-treated gravel.

Discussion of Findings Using Cement-Treated Gravel. The second test series in the evaluation of the effect of composition and width of loading strip was conducted on cement-treated gravel specimens. Results from this phase of testing are graphically illustrated in Figs 28 through 30. The data and the analysis of variance of the log-variance are summarized in Tables 4 and 5, respectively. The analysis of variance was conducted for strip type (steel and neoprene) and strip width (0.5, 1.0 inch).

The findings concerning indirect tensile strength are shown in Fig 28. It can be seen that neoprene loading strips and 1.0-inch width strips produce less dispersion of the data. The analysis of variance of the log-variance, however, indicates that these differences are not significant.

In the evaluation of vertical failure deformations (Fig 29) no correction was made for the deformation occurring in the steel strips; however, corrections were made for the deformation occurring in the neoprene. It may be noted that the standard deviations of the data were lower for both the neoprene





Analysis of Variance of Log-Variance

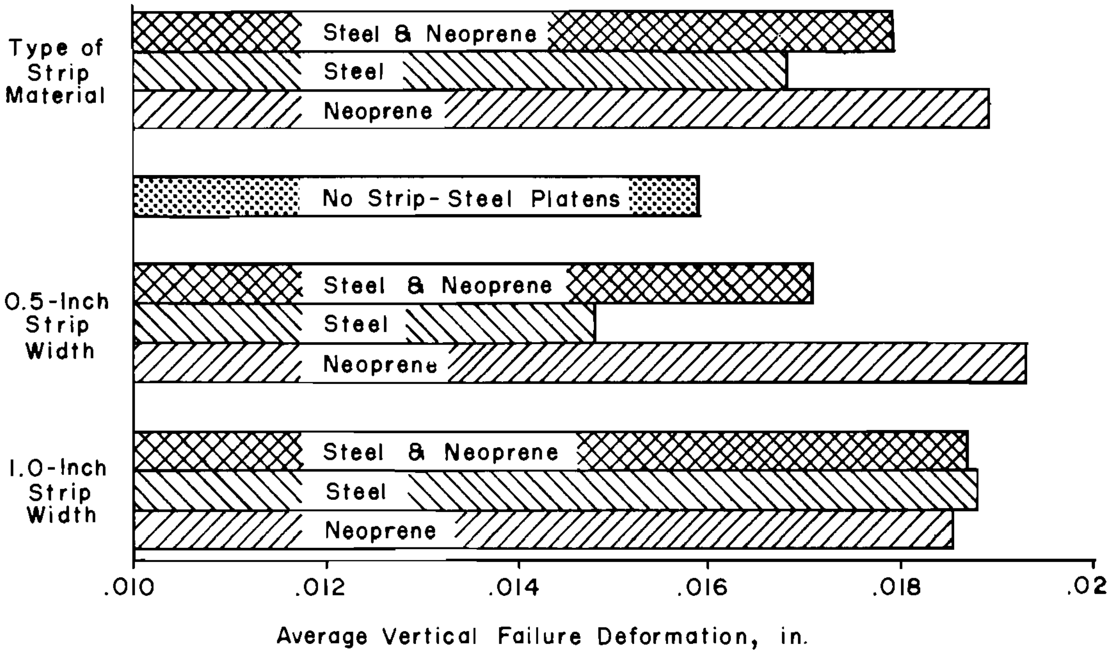
Source of Variation	Significance: Alpha Level	
Strip Type	—	None
Strip Width	—	None
Interaction	—	None

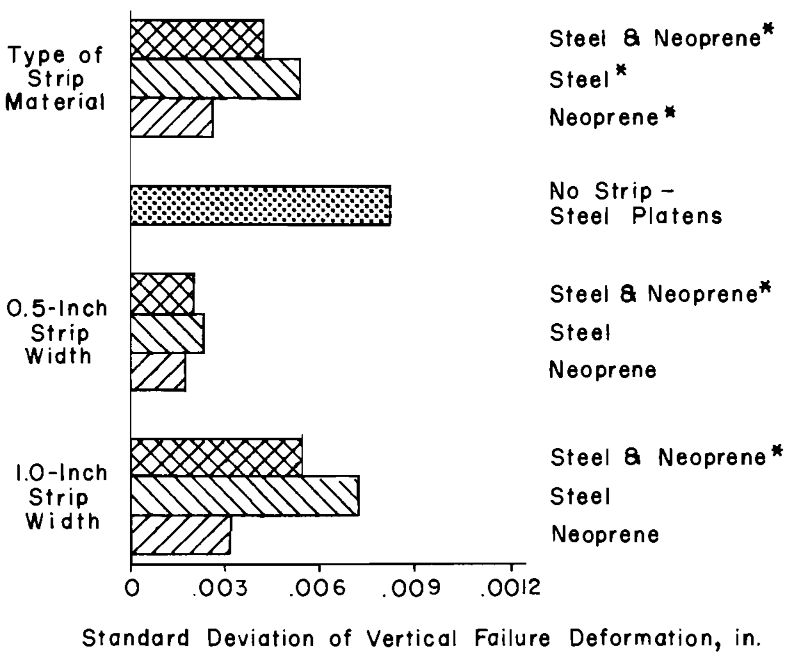
Loading Rate = 0.5 in/min

Testing Temperature = 75°

* Pooled Standard Deviation

Fig 28. Effect of type and width of loading strip on the dispersion and average values of indirect tensile strength for cement-treated gravel.





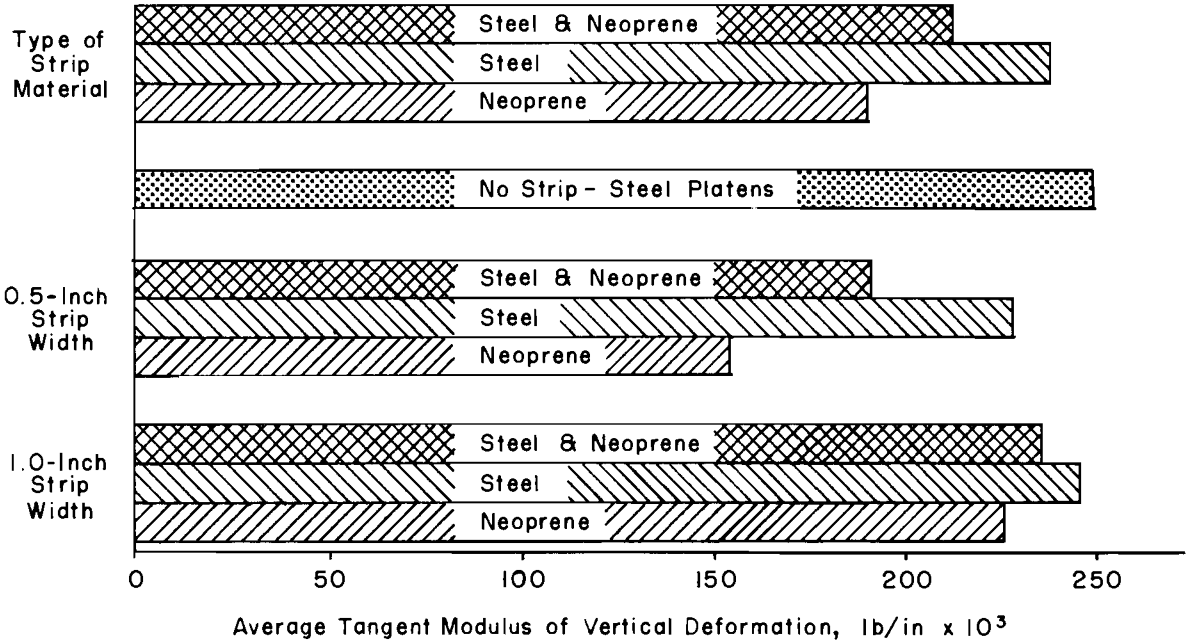
Analysis of Variance of Log-Variance		
Source of Variation	Significance	
	Alpha Level	
Strip Type	—	None
Strip Width	—	None
Interaction	—	None

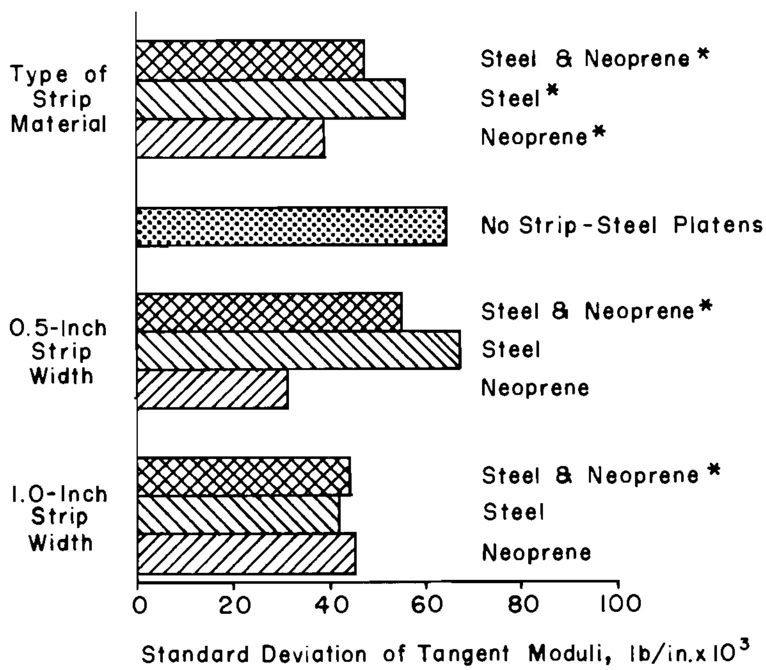
Loading Rate = 0.5 in/min

Testing Temperature = 75°

*Pooled Standard Deviation

Fig 29. Effect of type and width of loading strip on the dispersion and average values of vertical failure deformation for cement-treated gravel.





Analysis of Variance of Log-Variance		
Source of Variation	Significance	
	Alpha Level	
Strip Type	—	None
Strip Width	—	None
Interaction	—	None

Loading Rate = 0.5 in/min

Testing Temperature = 75° F

*Pooled Standard Deviation

Fig 30. Effect of type and width of loading strip on the dispersion and average values of tangent modulus of vertical deformation for cement-treated gravel.

TABLE 4. SUMMARY OF THE DATA FROM THE EVALUATION OF THE EFFECT OF STRIP TYPE AND WIDTH FOR CEMENT-TREATED GRAVEL

Type of Loading Strip		Neoprene		Stainless Steel		Platens (No Strips)
Strip Width, inches		0.5	1.0	0.5	1.0	∞
No. Specimens		5	5	5	5	5
Indirect Tensile Strength	Average, psi	146	178	166	177	167
	Std. Dev., psi	12.0	16.5	30.8	15.4	21.1
	Coef. of Var. %	8.3	9.3	18.6	8.7	12.6
Vertical Failure Deformation	Average, inches	.0193	.0186	.0148	.0188	.0159
	Std. Dev., inches	.00176	.00309	.00224	.00707	.00817
	Coef. of Var. %	9.1	16.7	15.2	37.6	51.4
Tangent Modulus of Vertical Deformation	Average, lb/in	154,680	225,260	227,660	245,910	248,880
	Std. Dev., lb/in	31,215	45,566	66,983	42,054	63,701
	Coef. of Var. %	20.2	20.2	29.4	17.1	25.6

TABLE 5. SUMMARY OF THE ANALYSIS OF VARIANCE OF THE LOG-VARIANCE OF THE EFFECT OF STRIP TYPE AND WIDTH FOR CEMENT-TREATED GRAVEL

	Source of Variation	Degrees of Freedom	Mean Squares	F	Significane Level %
Indirect Tensile Strength	Strip Type	1	1.936	1.64	None
	Strip Width	1	0.000	0.00	None
	Interaction	1	1.879	1.59	None
	Error	4	1.183		
Vertical Failure Deformation	Strip Type	1	0.3603	0.18	None
	Strip Width	1	0.2733	0.13	None
	Interaction	1	0.8006	0.39	None
	Error	4	2.0544		
Tangent Modulus of Deformation	Strip Type	1	1.5255	0.40	None
	Strip Width	1	0.0032	0.00	None
	Interaction	1	4.8996	1.28	None
	Error	4	3.8226		

strips and the 0.5-inch widths; these differences, however, were not significant as indicated by the analysis of variance of the log-variance.

In the analysis of the tangent modulus of deformation, illustrated in Fig 30, it was found that the neoprene strips and 1.0-inch width strips produced lower dispersion values than the steel strips and the 0.5-inch width strips. In the case of the steel strips the 1.0-inch strip produced the lower standard deviation while for the neoprene strips the lower value occurred with the 0.5-inch strip. The analysis of variance of the log-variance indicated these differences were not significant.

In summary it was found that steel loading strips resulted in higher strengths and lower failure deformations and, as a result, higher tangent moduli. In practically all cases the 1.0-inch strip resulted in a higher average parameter value than the 0.5-inch strip. The dispersion associated with the various testing conditions is of primary interest. Considering the effect of type of loading strip material, it generally was found that the standard deviations associated with the two materials were lower for the neoprene strips. With the exception of failure deformation the minimum dispersion occurred for the 1.0-inch loading strips. In all cases the differences in dispersion were slight and the analysis of variance of the log-variance showed no statistical significance.

The best strip appears to be neoprene as it did in the case of the test series on asphaltic concrete. Nevertheless, considering the very slight advantage of neoprene over steel which was not statistically significant and the large practical advantages of using steel strips, it is felt that the use of steel loading strips is justifiable. Analyzing the findings for only steel loading strips indicates that the 1.0-inch steel strips are better.

Recommendation Concerning the Composition and Width of Loading Strip. It is recommended that future testing utilize a loading strip composed of stainless steel which is 1.0 inch in width. This recommendation is based primarily upon the many practical advantages of using steel rather than the softer, more flexible neoprene. This recommendation is subject to change as more information is gathered and more materials are investigated.

Evaluation of the Effects of Testing Temperature and Loading Rate

The second phase of testing was concerned with the evaluation of the effects of testing temperature and loading rate. The evaluation was conducted on asphaltic concrete (Appendix A) because of its temperature susceptibility. Testing temperatures ranged from -10°F to 140°F ±2°F; loading rates ranged from 0.05 to 6.0 inches per minute. Testing utilized the electro-hydraulic, servo system shown in Fig 20 and one of the controlled temperature chambers available at The University of Texas. This chamber is capable of achieving temperatures ranging from -20°F to 140°F ±2°F and maintaining them for long periods of time. An experiment design (Fig 23C) with three blocks or replications was used in this phase of the testing. The analysis of variance of the log-variance is summarized in Table 6.

In Figs 31 and 32 it can be seen that the general shapes of the strength-temperature and tangent modulus-temperature relationships are similar. In these figures a substantial change occurs in the slope of the relationships at or slightly less than 80°F indicating that the effects of temperature are much more significant in the lower temperature range. It may also be noted that at the lower temperatures the relationships become somewhat erratic. Maximum vertical failure deformation (Fig 33) occurred at some intermediate temperature of about 80°F. Minimum failure deformation occurred at -10°F for all loading rates, illustrating the stiffness of the mix at low temperatures.

Examination of Figs 34 through 36, however, indicates that the effect of load rate is not as great as the temperature effect. A possible exception can be seen for the strength averages obtained at very low loading rates. There would appear to be a substantial increase in the mean value as the loading rate is increased at these low rates, especially at low testing temperatures.

The effect of temperature and of loading rate on the dispersion of the test data is illustrated in Figs 37 and 38, respectively. Figure 37 indicates a substantial temperature effect on the dispersion of the strength and tangent modulus values with the dispersion decreasing as the temperature increases. There would appear to be no temperature effect on the dispersion of the vertical failure deformations and little if any effect due to loading rate. Analysis of variance of log-variance substantiates these observations by showing a highly significant ($\alpha = 0.01$) temperature effect on the variance of the strengths and the tangent moduli but no significant temperature effect for vertical deformations and no significant effect due to changes in loading rate. The reduction in variance of strength in the range between 50° and 80°F (Fig 37) is statistically significant ($\alpha = 0.05$).

TABLE 6. ANALYSIS OF VARIANCE OF THE LOG-VARIANCE OF THE EFFECT OF TESTING TEMPERATURE AND LOADING RATE FOR ASPHALTIC CONCRETE

	Source of Variation	Degrees of Freedom	Mean Square	F	Significance Level, %
Indirect Tensile Strength	Temperature	5	6.9171	26.70	1
	Loading rate	5	0.4879	1.89	None
	Residual	25	0.2591		
Vertical Failure Deformation	Temperature	5	0.0791	0.25	None
	Loading rate	5	0.2379	0.74	None
	Residual	25	0.3214		
Tangent Modulus of Vertical Deformation	Temperature	5	10.7246	26.72	1
	Loading rate	5	0.6861	1.71	None
	Residual	25	0.4014		

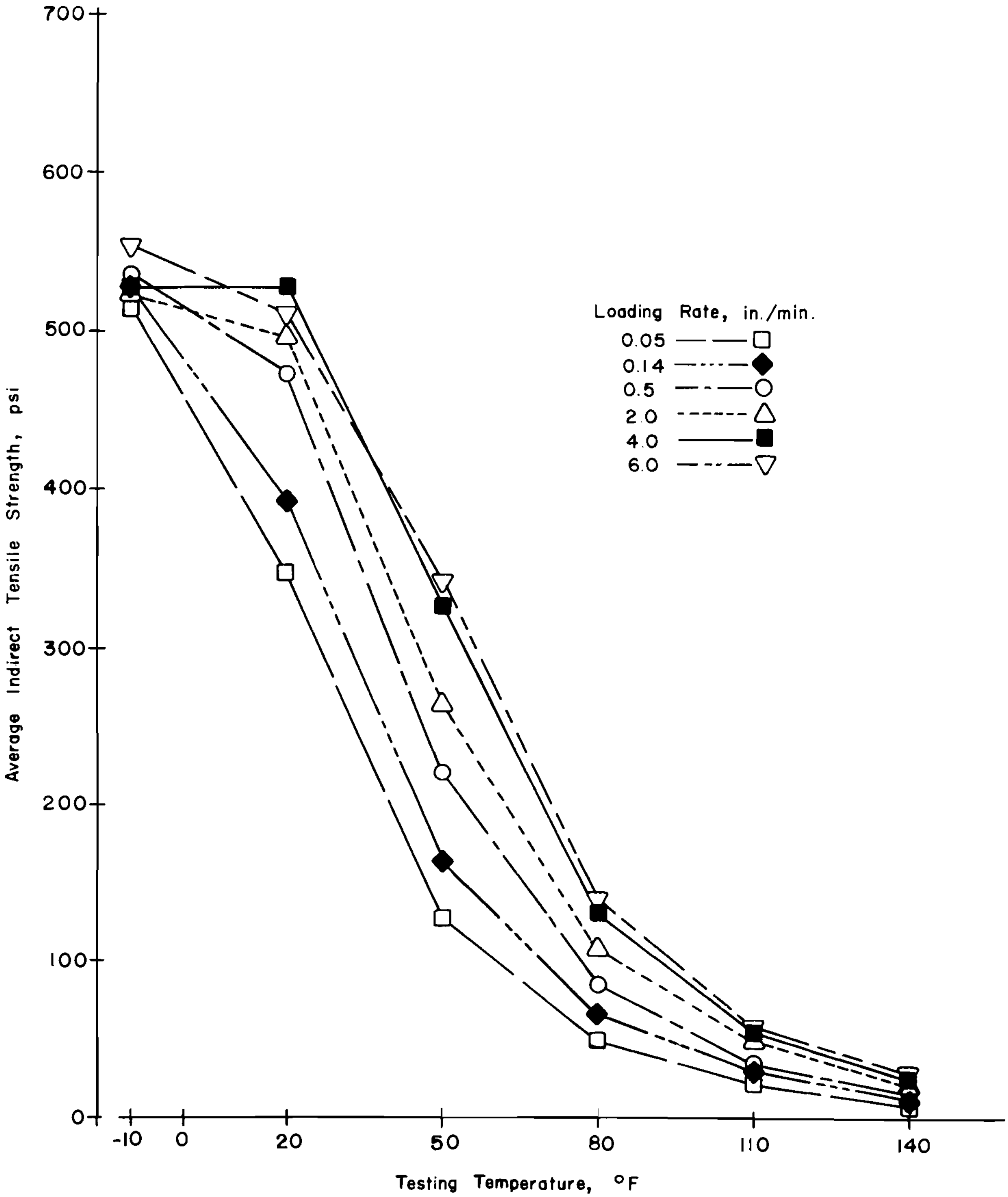


Fig 31. Effect of testing temperature on indirect tensile strength for asphaltic concrete.

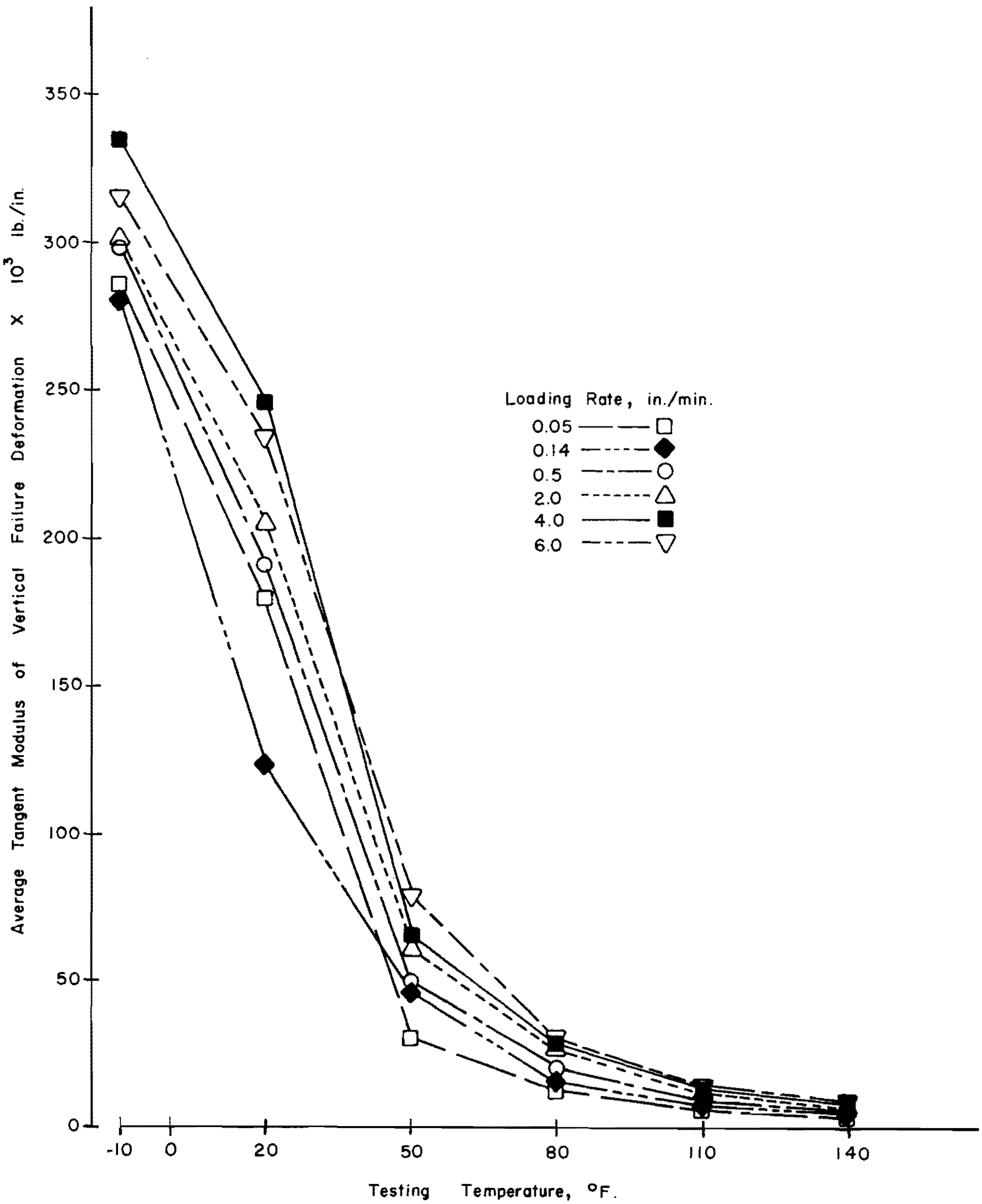


Fig 32. Effect of testing temperature on tangent modulus of vertical deformation for asphaltic concrete.

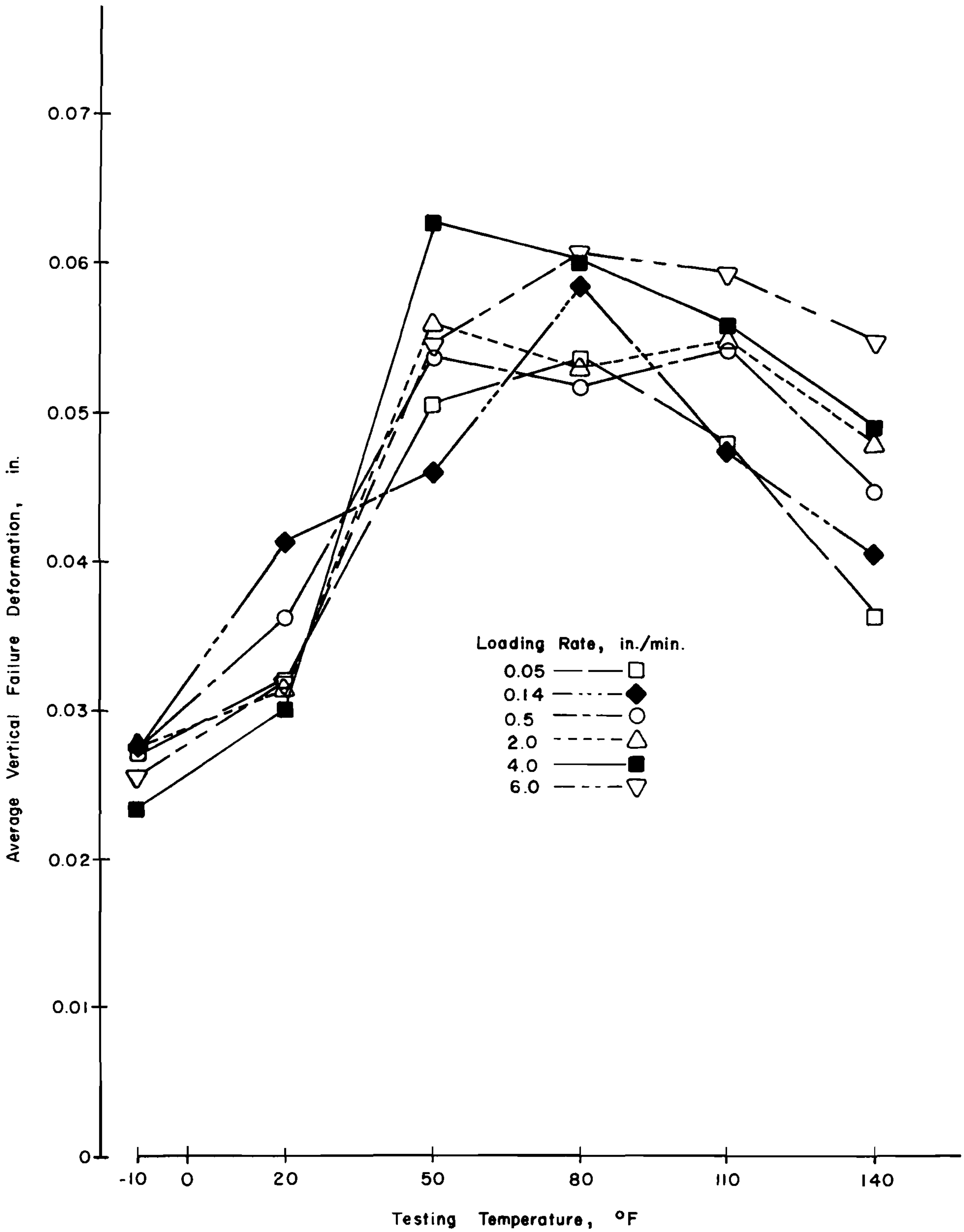


Fig 33. Effect of testing temperature on vertical failure deformation for asphaltic concrete.

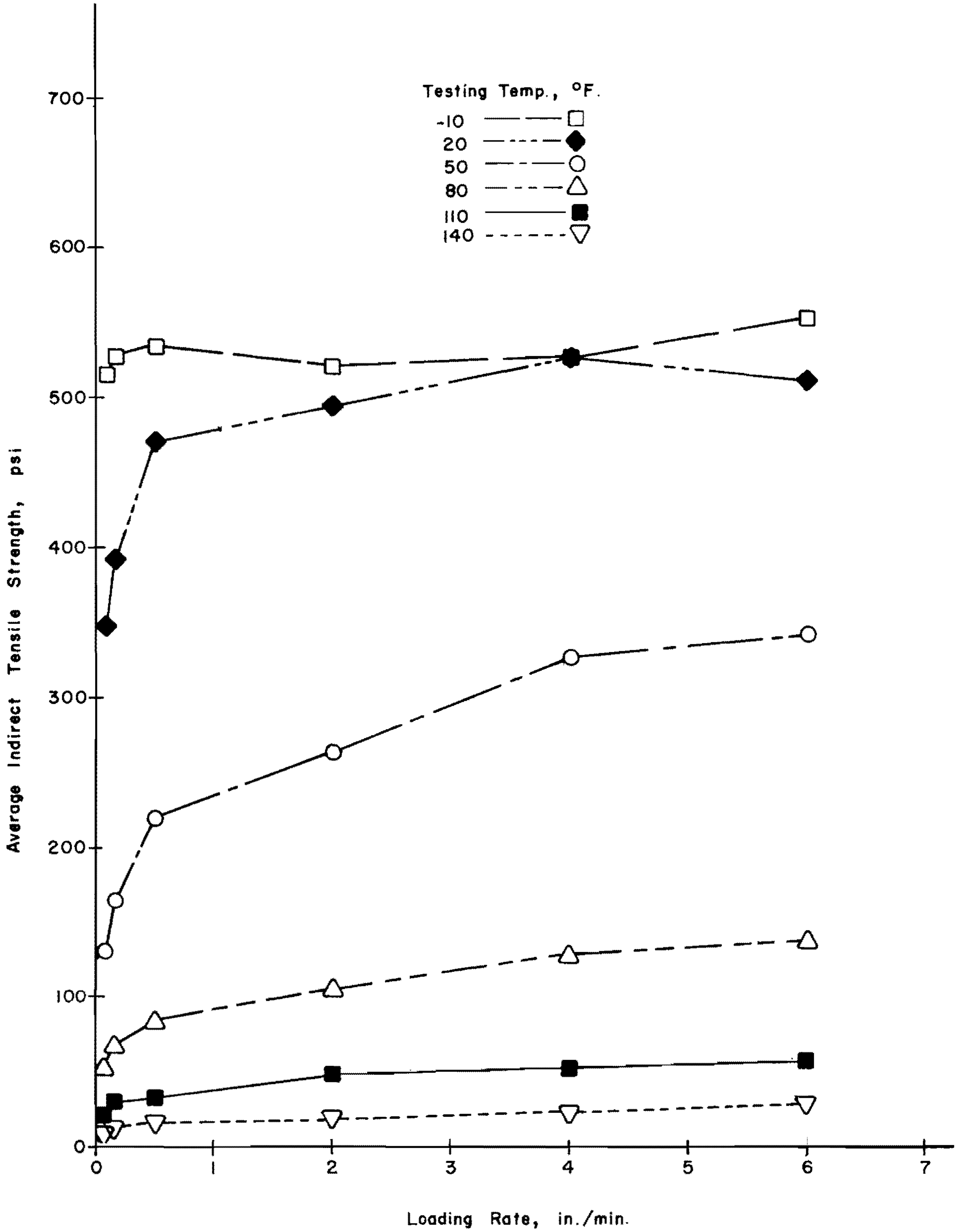


Fig 34. Effect of loading rate on indirect tensile strength for asphaltic concrete.

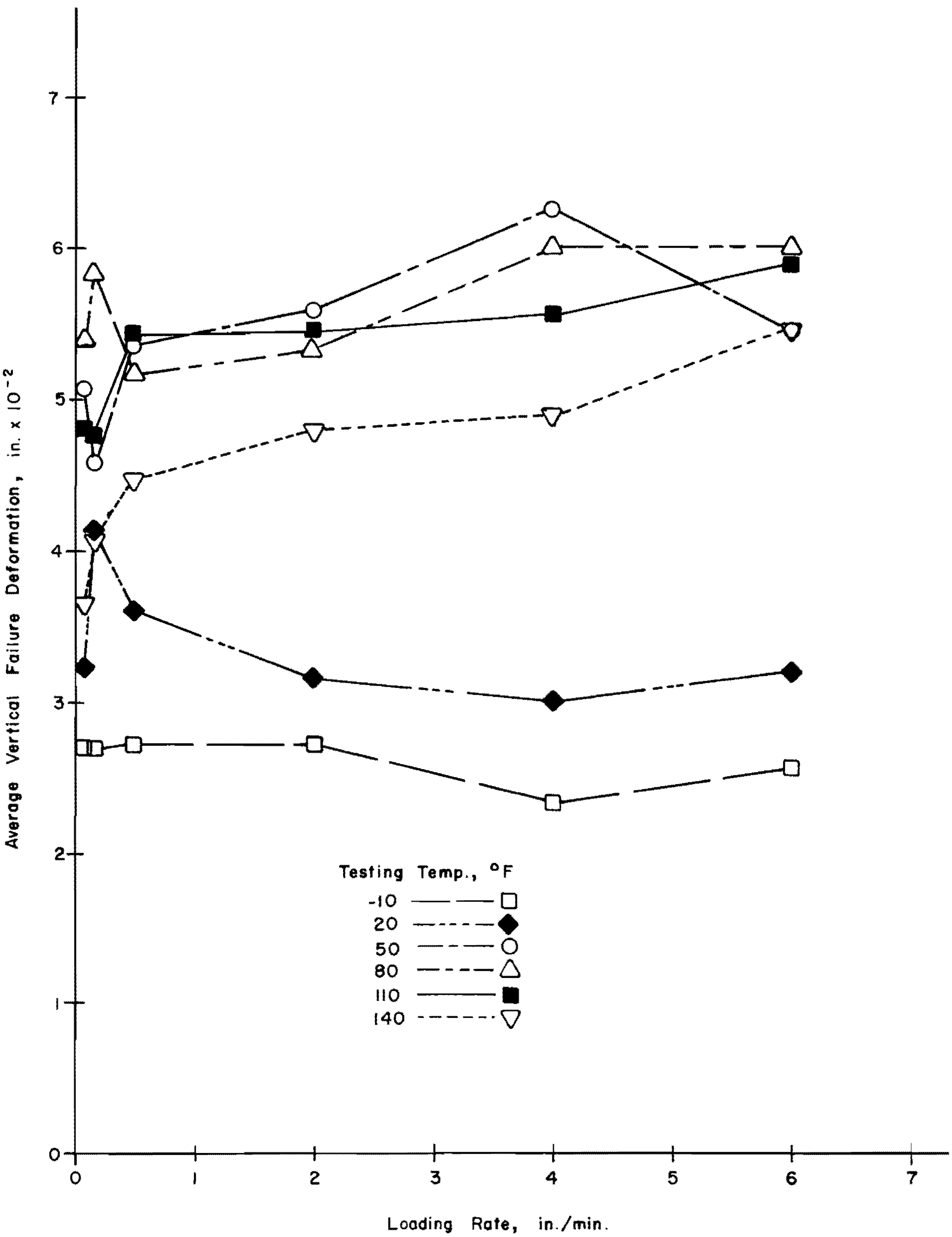


Fig 35. Effect of loading rate on vertical failure deformation for asphaltic concrete.

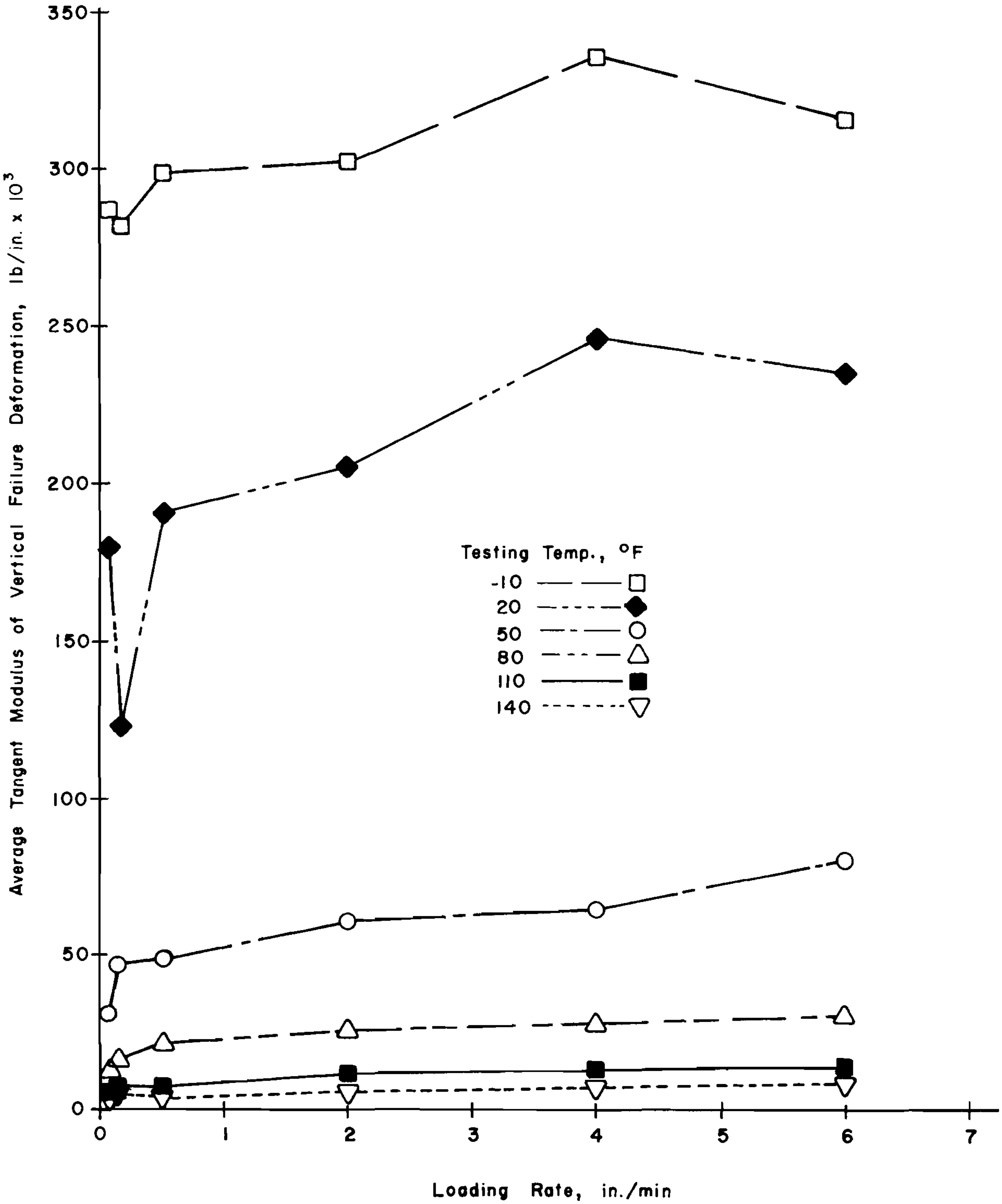


Fig 36. Effect of loading rate on tangent modulus of vertical deformation for asphaltic concrete.

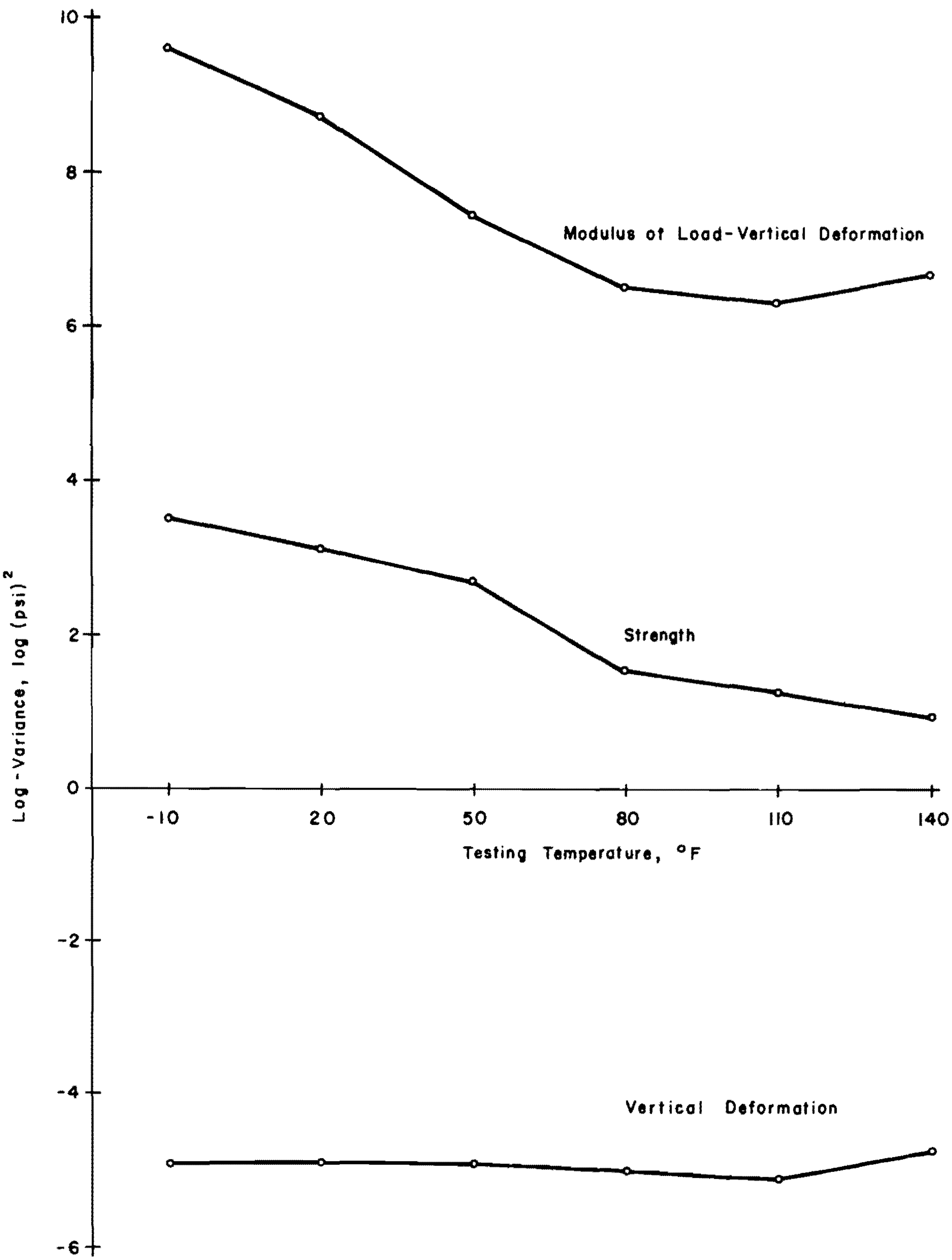


Fig 37. Effect of testing temperature on the log-variance of strength, vertical deformation, and the load-deformation modulus of asphaltic concrete.

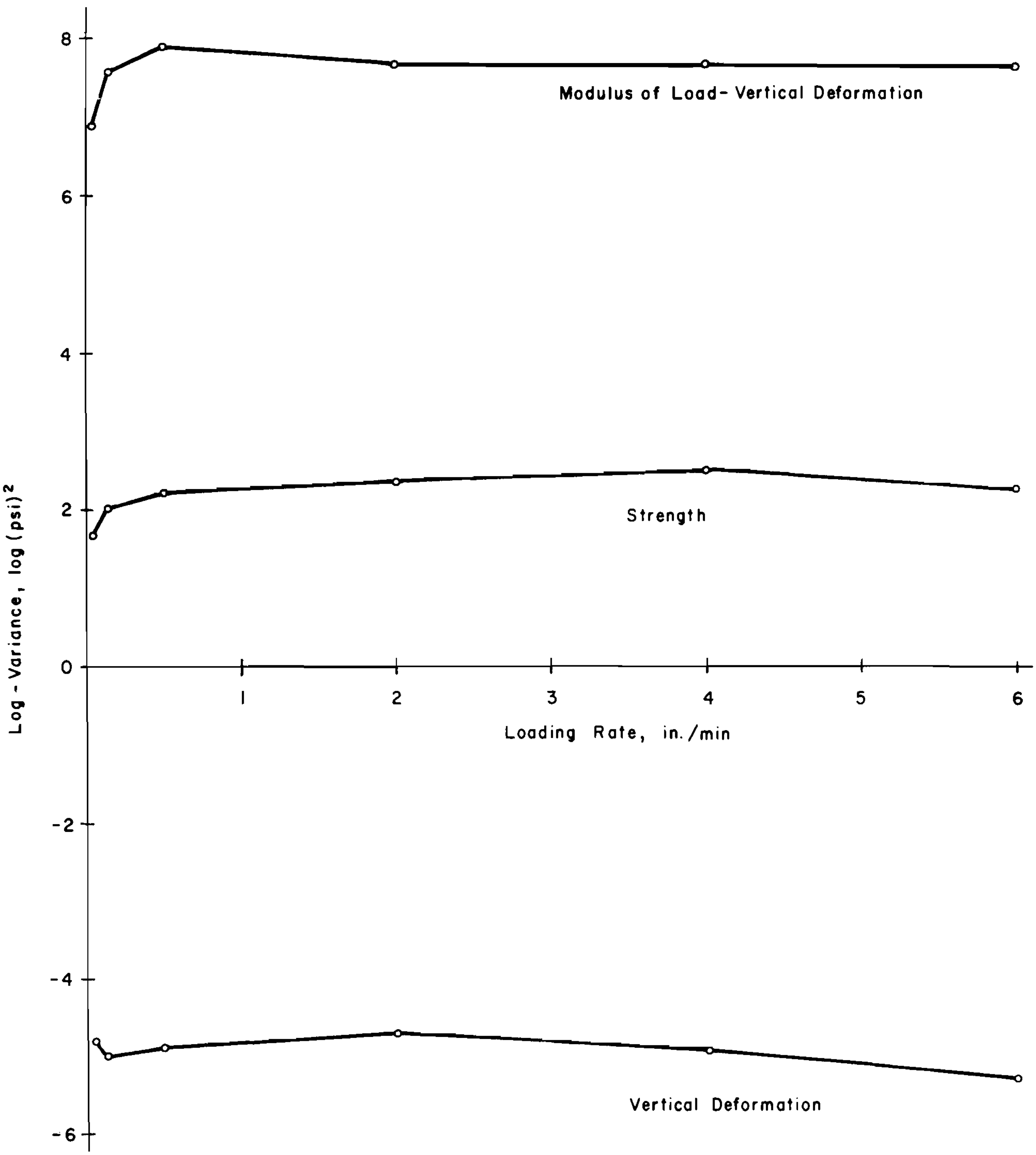


Fig 38. Effect of loading rate on the log-variance of strength, vertical deformation, and the load-deformation modulus of asphaltic concrete.

It is recommended that future testing be conducted at room temperature (77°F) at a loading rate of 2.0 inches per minute. This temperature was chosen because (1) it approximates the lower temperature range in which the strength and tangent-modulus parameters were relatively uniform and nontemperature susceptible, (2) it approximates the lower limit of the temperature range exhibiting reasonably low dispersion values for strength and to a certain extent, tangent modulus, (3) it has previously been used as a standard testing temperature, and (4) it is fairly close to the normal temperature of air conditioned laboratories and, thus, does not require special equipment or facilities for substantially raising or lowering the temperature. The loading rate of 2.0 inches per minute was chosen primarily as a compromise. At slow loading rates the magnitudes of the test parameters were more susceptible to loading-rate changes than at higher rates. In addition, the theory assumes a linear stress-strain or brittle characteristic for the material being tested, and a more rapid loading rate tends to produce a more brittle behavior in the tested material. At the very rapid loading rates, however, the test is more difficult to conduct. At 2.0 inches per minute the indrect tensile test was easy to conduct, and this loading rate is above the range in which the test parameters appeared to be very susceptible to changes in loading rate.

<u>Evaluation of Mode of Failure</u>

A basic requirement of the indirect tensile test is that the specimen fail in tension rather than in compression or shear. In order to ascertain whether the specimens tested during this experimental program failed in tension, a cursory examination was made of the failure pattern on every specimen. No cases of shear or compressive failure were detected.

Failures generally appeared in one of 3 forms, localized crushing (Fig 19b), single cleft (Fig 19d), and double cleft (Fig 19c). Asphaltic concrete specimens tested at room temperature (75°F) exhibited a limited amount of localized crushing, while the more brittle cement-treated specimens tested at room temperature often failed with a single or double cleft pattern. The failure patterns observed in the asphaltic concrete series with temperatures ranging from -10°F to 140°F varied from localized crushing at 140°F to double and single cleft failures at -10°F.

CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

(1) Review of existing information indicates that the indirect tensile test is the best test currently available for determining the tensile properties of highway materials.

(2) From this information and the results of a limited testing program the indirect tensile test appears to be a feasible method for evaluating the tensile characteristics of stabilized subbase materials although previous use of this test has generally been with concrete.

(3) Primary characteristics of the indirect tensile test and the materials tested which may affect the test results are

  (a) load-deformation characteristics of the material tested,
  (b) size and dimensions of the specimen,
  (c) composition and dimensions of the loading strip,
  (d) rate of loading, and
  (e) testing temperature.

(4) Characteristics and properties of the material being tested are not considered in the theoretical development of the test, except as a limiting tensile strength. The materials are assumed to have linear-elastic stress-strain characteristics. Although many deviations from the assumed conditions exist and although the use of the simple formula $S_T = \frac{2P_{max}}{\pi t d}$ introduces small errors in the results, there does not appear to be any evidence that the error is significant as long as the specimen ultimately fails in tension.

(5) The indirect tensile strength has been shown both theoretically and experimentally to be independent of the length-diameter ratio. It has been assumed that other indirect tensile parameters such as failure deformations and load-deformation characteristics are also independent of this ratio.

(6) The indirect tensile strength is reduced slightly by an increase in the overall size of the specimen, and the dispersion of the data is reduced.

(7) On the basis of the literature review it is concluded that the composition and width of the loading strip have a definite effect the stress distribution in the specimen, the test results, and the mode of failure.

(8) Wood which has often been recommended as a loading strip was eliminated from future use by this project because of practical difficulties associated with measuring deformations in the specimen.

(9) It is recommended that steel be used as a loading strip because of its significant practical advantages even though experimental results presented in this paper indicate that neoprene is a slightly, but not significantly, better loading strip material than steel.

(10) A one-inch-wide strip is recommended over a half-inch width because of the reduced data dispersion.

(11) Under the conditions of the tests performed in this study temperature had a highly significant effect on the dispersion of the strength and load deformation moduli; however, the dispersion of the vertical failure deformation was not significantly affected by temperature. It was also noted that there was a statistically significant reduction in the dispersion of the strength data in the temperature range of 50° to 80°F.

(12) Under the conditions of the tests performed in this study loading rate had no significant effect on the dispersion of the results.

RECOMMENDATIONS

Based on the conclusions stated above certain decisions concerning parameters in the indirect tensile test have been made. The parameters will be fixed tentatively for evaluation tests to be conducted in the project in the near future.

(1) The specimen will be as large as is practical in order to obtain more uniform test results and a better measure of the average of the test results. It is planned that ultimately samples will be 6 inches in diameter with heights in the range of 8 to 12 inches.

(2) The loading strip will be stainless steel with a width of one inch.

(3) The loading rate will be 2.0 inches per minute.

(4) The testing temperature will be room temperature in the range of 75 to 77°F.

This page replaces an intentionally blank page in the original.
-- CTR Library Digitization Team

REFERENCES

1. Westergaard, H. M., "Analytical Tools for Judging Results of Structural Tests of Concrete Pavements," Public Roads, Vol 14, 1933.

2. Westergaard, H. M., "Computation of Stresses in Concrete Roads," Proceedings, Fifth Annual Meeting of the Highway Research Board, 1926.

3. Hudson, W. Ronald, and Hudson Matlock, "Discontinuous Orthotropic Plates and Pavement Slabs," Research Report Number 56-6, Center for Highway Research, The University of Texas, Austin, Texas, May 1966.

4. Hudson, W. R., "Discontinuous Orthotropic Plates and Pavement Slabs," Ph.D. Dissertation, The University of Texas, Austin, Texas, August 1965.

5. "Stresses and Deflections in Foundations and Pavements", Soil Mechanics and Bituminous Materials Research Laboratory, Institute of Transportation and Traffic Engineering, University of California, Berkeley, 1963.

6. Burmister, Donald M., "Evaluation of Pavement Systems of the WASHO Road Test by Layered System Methods," Bulletin 177, Highway Research Board, publication 537, 1958, pp 26-54.

7. Burmister, Donald M., "Stress & Displacement Characteristics of a Two-Layered Rigid Base Soil System: Influence Diagrams and Practical Applications," Proceedings of Highway Research Board, 1956.

8. The AASHO Road Test: Report 5 - Pavement Research, Special Report 61E, Highway Research Board, publication No 954, 1962.

9. Nussbaum, P. J., and T. J. Larsen, "Load Deflection Characteristics of Soil-cement Pavements," Highway Research Record, No 86, Highway Research Board, 1965; Bulletin D96, Portland Cement Association Development Department, Skokie, Illinois.

10. Childs, L. D., "Cement-Treated Subbases for Concrete Pavements," Tentative Report No 1249, Portland Cement Association Research and Development Laboratories, Skokie, Illinois, September 1966.

11. Williamson, R. S., "A Critical Look at Stabilization," a paper presented at the Fortieth Annual Texas Highway Shortcourse, December 1966.

12. Messina, R., "Split Cylinder Test for Evaluation of the Tensile Strength of Asphalt Concrete Mixtures," unpublished Thesis, Master of Science in Civil Engineering, The University of Texas, Austin, Texas, January 1966.

13. Mitchell, N. B., Jr., "The Indirect Tension Test for Concrete," Materials Research and Standards, ASTM, Vol 1, No 10, October 1961, pp 780-787.

14. ASTM Standard for Concrete, C78-64, Part 10, 1964, p 37.

15. ASTM Standard for Concrete, C2983-64, Part 10, 1964, p 212.

16. Grieb, W. E., and G. Werner, "Comparison of the Splitting Tensile Strength of Concrete with Flexural and Compressive Strengths," Public Roads, Vol 32, No 5, December 1962, pp 97-106.

17. Thaulow, Sven, "Tensile Splitting Test and High Strength Concrete Test Cylinders," Journal of the American Concrete Institute, Vol 28, No 7, Paper 53-38, January 1957, pp 699-705.

18. Jimenez, R. A., "Evaluation of the Cohesiometer Test for Asphaltic Concrete," Highway Research Record, No 51, Highway Research Board, 1964.

19. Carniero, F. L. L. B., and A. Barcellos, "Concrete Tensile Strength," Bulletin No 13, International Association of Testing and Research Laboratories for Materials and Structures, Paris, March 1953, pp 97-127.

20. Akazawa, T., "Tension Test Method for Concrete," Bulletin No 16, International Association of Testing and Research Laboratories, Paris, November 1953, pp 11-23.

21. Thompson, M. R., "The Split-Tensile Strength of Lime-Stabilized Soils," Lime Stabilization, Highway Research Record, No 92, Highway Research Board, 1965, pp 69-79.

22. Breen, J. J., and J. E. Stephens, "Split Cylinder Test Applied to Bituminous Mixtures at Low Temperatures," Journal of Materials, Vol 1, No 1, American Society for Testing and Materials, March 1966.

23. Breen, J. J., and J. E. Stephens, "Fatigue and Tensile Characteristics of Bituminous Pavements at Low Temperatures," Report No JHR 66-3, School of Engineering, University of Connecticut, July 1966.

24. Livneh, M., and E. Shklarsky, "The Splitting Test for Determination of Bituminous Concrete Strength," Proceedings, AAPT, Vol 31, 1962, pp 457-476.

25. Hertz, H., "Uber die Verteilung der Druckkrafte in einem elastischen Kreiscylinder," Zeitschrift fur Mathematik and Physik, Vol 28, 1883.

26. Foppl, A., and L. Foppl, Drang und Zwang, V. I., Oldenburg Verlag, Berlin, 1941.

27. Timoshenko, S., and J. N. Goodier, "Concentrated Force at a point of a Straight Boundary," Theory of Elasticity, Second Edition, McGraw-Hill, New York, 1951, p 85.

28. Timoshenko, S., and J. N. Goodier, "Stresses in a Circular Disk," Theory of Elasticity, Second Edition, McGraw-Hill, New York, 1951, p 107.

29. Frocht, M. M., Photoelasticity, Vol 2, John Wiley & Sons, Inc., New York, 1957.

30. Peltier, R., "Theoretical Investigation of the Brazilian Test," Bulletin No 19, Union of Testing and Research Laboratories for Materials and Structures, Paris, November 1954, pp 31-69.

31. Wright, P. J. F., "Comments on an Indirect Tensile Test on Concrete Cylinders," Magazine of Concrete Research, Vol 7, No 20, London, July 1955, pp 87-96.

32. Rudnick, A., A. R. Hunter, and F. C. Holden, "An Analysis of the Diametral-Compression Test," Materials Research & Standards, ASTM, Vol 3, No 4, April 1963, pp 283-289.

33. Ramesch, C. K., and S. K. Chopra, "Determination of Tensile Strength of Concrete and Mortar by the Split Test," Indian Concrete Journal, Vol 34, 1960, pp 354-357.

34. Bawa, N. S., "The Analysis of Stress Distribution in Indirect Tensile Tests on Concrete Cylinders," Indian Concrete Journal, March 15, 1957, pp 90-92.

35. Efsen, A., Glarbo, O., "Tensile Strength of Concrete Determined by Cylinder Splitting Test," Beton og Jernbeton, No 1, Copenhagen, Denmark, 1956, pp 33-39.

36. Thaulow, Sven, Betongkontroll paa byggeplassen, Norsk Cementforening, Oslo, Norway, 1953, p 224; Field Testing of Concrete, abridged version in English, p 64.

37. Addinall, E., and P. Hackett, "The Effect of Platen Conditions on the Tensile Strengths of Rock-like Materials," Civil Engineering and Public Works Review, October 1964.

38. Simon, Leslie L., and M. I. Aust, "The Use of the Indirect Tensile Test for Quality Control of Pavement Concrete," Construction Review, August 1956, pp 23-29.

39. Igbal Ali, E. Gruenwald, H. Rusch, and G. Vigerust, "Discussion," Journal of the American Concrete Institute, Vol 29, No 6, Paper 53-38, December 1957.

40. ASTM Tentative Standard for Concrete, C496-64T, Part 10, p 357.

This page replaces an intentionally blank page in the original.
-- CTR Library Digitization Team

APPENDIX A

MIX DESIGN AND SAMPLE PREPARATION PROCEDURE FOR ASPHALTIC CONCRETE

This page replaces an intentionally blank page in the original.
-- CTR Library Digitization Team

APPENDIX A

MIX DESIGN AND SAMPLE PREPARATION PROCEDURE FOR ASPHALTIC CONCRETE

Aggregate: Crushed limestone

Asphalt: AC-10

Water, %	Nil
Viscosity at 275° F , Stokes	2.6
Viscosity at 140° F , Stokes	1088
Solubility in CCL_4, %	99.7+
Flash Point C. O. C., °F	570
Ductility, 77° F , 5 cm/min, cm	141+
Viscosities Determined at 77° F	4.0
Penetration at 77° F , 100 g, 5 sec . . .	92
Specific Gravity at 77° F	1.006

Design: Asphalt Content: 5.3% by wt.

Gradation: Texas Highway Department gradation Item 340, Type D

Sieves	%, by weight
1/2" - 3/8"	2
3/8" - No. 4	38
No. 4 - No. 10	23
No. 10 - No. 40	16
No. 40 - No. 80	12
No. 80 - No. 200	7
Passing No. 200	2

Specimen Size: 4" diameter × 2" height

Sample Preparation:

(1) Mixed at 275° F ± 5° F for 3 minutes in an automatic, 12-qt capacity Hobart food mixer at 107 rpm.

(2) Cured at 140° F ± 5° F for 18 to 24 hours.

(3) Compacted at 250° F ± 5° F.

Compaction:

Gyratory shear compaction performed according to Texas Highway Department Standard 206-F, Part II.

Testing Procedure:

(1) Pre-heat at 180° F for 18-24 hours.

(2) Cool at 75° F for 18-24 hours.

(3) Hold at testing temperature for 18-24 hours prior to testing.

APPENDIX B

MIX DESIGN AND SAMPLE PREPARATION PROCEDURE FOR CEMENT-TREATED MATERIAL

This page replaces an intentionally blank page in the original.
-- CTR Library Digitization Team

APPENDIX B

MIX DESIGN AND SAMPLE PREPARATION PROCEDURE FOR CEMENT-TREATED MATERIAL

Aggregate: Rounded gravel

Wet Ball Mill - 37.2%

Los Angeles Abrasion (100 rev) - 7.2%

(500 rev) - 27.2%

Cement: Portland Cement, Type I,

Design: Cement content: 6% by wt.

Moisture content: 6% by wt.

Gradation: Texas Highway Department Gradation Item 340, Type D

Sieves	% by Wt
1/2" - 3/8"	2
3/8" - No. 4	38
No. 4 - No. 10	23
No. 10 - No. 40	16
No. 40 - No. 80	12
No. 80 - No. 200	7
Passing No. 200	2

Specimen size: 4" dia. × 2" ht.

Sample preparation:

(1) Prepared according to Texas Highway Department Test Method Tex-120E.

(2) Cured with top and bottom porous stones in place for 7 days in moisture room.

Compaction:

Gyratory shear compaction: performed according to a modification of Texas Highway Department Test Method Tex-206F, Part II. Modification: compaction process is terminated when 150 lb pressure is achieved <u>during</u> gyration.

Testing procedure:

(1) Remove from moisture room and drain excess water.

(2) Test at room temperature (75 to 77°F).