Analysis of Flexible Paving Mixtures by Theoretical Design Procedure Based on Shear Strength

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Instrumentation for the preparation and testing of flexible paving mixtures in flexure, tension, and triaxial compression is described. The influence of test temperature and rate of loading on asphalt concrete is developed. Mixture variables in the investigation include aggregate gradation, asphalt content, penetration and viscosity of asphalt, binder blend using flake asphalt and polyethylene, Viadon binder content, and asbestos fibers in mixture. Mixtures are analyzed for suitability to resist compressive stresses in the pavement surface course. The practical application of this procedure to design is shown.

•THE ANALYSIS or design of a mixture for the surface course of a flexible pavement should consider the shear strength properties of the mixture under critical service conditions. Present standard test procedures for the strength of mixtures are empirical and, as such, preclude theoretical analysis. The surface of a flexible pavement structure in service is subjected to flexural, tensile, and compressive stresses. The determination of the resistance of paving mixtures to these stresses for purposes of design necessitates the development of suitable instrumentation for the preparation and testing of test specimens. During the past 2 years, the author designed equipment for the testing of mixtures in flexure, tension, and compression, including the triaxial cell. It is the purpose of this paper to present the instrumentation and test results obtained for various mixtures under varying conditions of test. An analysis of these mixtures by an equation (1, 2) based on shear strength is included. The application to design practice is introduced.

FACTORS IN DESIGN OF FLEXIBLE PAVING MIXTURES

Shear Strength

The resistance which a flexible pavement component offers to plastic displacement from the application of wheel loads depends on the shear strength of the component. Shear strength is developed through intergranular friction as measured by the angle of internal friction, ϕ, and through the internal resistance of the binder to shear as measured by cohesion, c. These stress constants have been determined graphically from Mohr diagrams for this presentation, following an established procedure.

Internal Friction.—This factor is influenced by the density of the mix and by the properties of the aggregate such as hardness, surface texture and shape. Density of mix is a function of (a) aggregate gradation, (b) amount and viscosity of asphalt and (c) compactive effort. For a given gradation of aggregate there is an optimum asphalt content at which the mix will produce maximum density for a given compactive effort. An increase in compactive effort tends to reduce the optimum asphalt content. For a given aggregate and binder content the internal friction, ϕ, varies with density of mineral aggregate. Of primary importance in the design of asphalt mixes is the gradation of the mineral aggregate.

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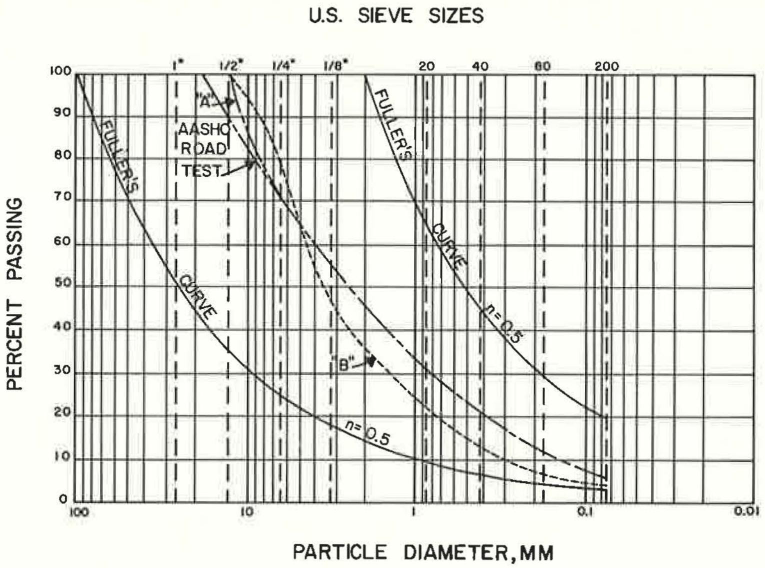

Figure 1. Gradation curves for aggregate used in paving mixtures.

Soon after the beginning of the present century, Fuller showed that aggregate will have a relatively high density when its particle size distribution follows the equation

$$P = 100\left(\frac{S}{M}\right)^n \tag{1}$$

where

P = percent of total aggregate passing any particular sieve,
S = size along side of sieve opening,
M = size of largest piece of aggregate, and
n = a constant.

Since that time Fuller's findings have been widely accepted, with most authorities agreeing that the value of n is approximately 0.5 for both soils and aggregates. The value of 0.5 has become popular, mainly because of the ease with which it may be used. When plotted on a semilogarithmic graph having a vertical arithmetic scale representing P and a horizontal logarithmic scale representing S, Fuller's equation becomes a sagging curve. Figure 1 gives two such curves, along with gradations A and B as used in this investigation.

Fuller's equation becomes a straight line when drawn on a graph having logarithmic scales along both axes. To show how this is possible, we take the logarithms of both sides of Eq. 1, which gives

$$\log P = \log 100 + n \log S - n \log M = K + n \log S \tag{2}$$

where K is some constant which depends only on the size of the largest piece of aggregate. Eq. 2 is in the form of the equation for a straight line having customary x and y axes ($y = ax + b$) if $y = \log P$, $a = n$, $x = \log S$, and $b = K$.

Mixes which plot with a straight-line slope of 0.5 on the log-log graph are very tight and optimum asphalt content is on the low side (4.0 to 5.0 percent for ½-in. maximum size aggregate). Such mixes are difficult to work and would probably present problems in construction. This agrees with work by McLeod (3) in his analysis of void requirements for dense-graded paving mixtures. The plot for gradation B approaches a straight line but has a slope of about 0.65.

Cohesion.—This mix property is dependent on the asphalt binder and is developed through the adhesion of the binder to the aggregate and through the internal resistance of the asphalt to deformation. The amount of cohesion developed in an asphalt mix is influenced by: (a) amount of asphalt cement in the mix, (b) thickness of asphalt film, (c) rate of load application, and (d) temperature of the mix. Viscous resistance is influenced by the rate of deformation; i. e., the higher the rate of application the greater the resistance. The AASHO Road Test showed that static load deflections were about twice those at 50 mph (4). Asphalt is also more viscous at low than at high temperatures. On the basis of shear strength, high summer temperatures are critical and it would be at this time that surface-originated deformations develop.

Voids in Mix

The voids in an asphalt paving mixture are controlled by specifying the range of air voids in the mixture (referred to as voids in total mix), prescribing the range for voids in mineral aggregate which is to be filled with asphalt, and controlling the voids in the mineral aggregate. McLeod (3) has stated that a minimum of 15 percent voids should be provided in the mineral aggregate to assure adequate space for the asphalt binder. This is controlled by aggregate gradation and normally requires some deviation from the Fuller curve ($n = 0.5$). The requirement for air voids in total mix is about 3 to 5 percent with 75 to 85 percent of the aggregate voids filled with asphalt. It is generally a stipulation that voids be determined on the basis of apparent specific gravity of aggregate; however, some procedures require the bulk specific gravity and McLeod (3) suggests that an effective specific gravity should be employed which gives values between apparent and bulk. The problem of selection of the appropriate specific gravity relates to permeable voids in the surface of the aggregate and would be a serious problem only for rather porous aggregates. The purpose of establishing a minimum of air voids in the mix is to provide some space for in-service compaction and to prevent loss of internal friction, ϕ, through lubrication, thus causing excessive surface deformation and possible bleeding of the surface. A maximum permissible air voids content is stipulated to insure a mix of low permeability and to reduce hardening of the asphalt binder by oxidation and loss of volatiles.

Flexibility

Flexibility is a measure of the magnitude of flexural deflection which a component of a flexible pavement structure can withstand without fracture. For more permanent flexible pavements, the deflections should be small. The best flexible pavement is one which produces the least flexural strains. A flexible pavement is designed to resist shear stresses; a rigid pavement is designed to resist flexural stresses. The term flexible should not be taken too literally in the design of asphalt mixtures. The difference in the magnitude of deflection for properly designed rigid and flexible pavements is small.

The requirement for flexibility in an asphalt surface mixture is influenced by the total pavement structure. Flexure failures are normally related to low temperatures of the surface and high moisture content in the subgrade or road base structure. Some flexure cracking may develop through fatigue failure, which is influenced by the magnitude of deflection and the viscosity and amount of binder. It has been shown (6) that for high deflection a thin pavement section can withstand flexural fatigue better than thick sections. Consequently, if a pavement structure is to permit high surface deflections, it might be advisable to provide a thin surface course in design.

Surface Texture

The design of a surface mixture would not be complete without consideration of the non-skid properties and abrasion resistance of the surface. It may be possible to design a mixture satisfying the requirements for shear strength, voids, and flexibility with aggregate of ⅛-in. maximum size; however, this mixture probably would not provide the skid resistance required on heavy-duty rural roads. The maximum size of aggregate specified for a mixture, and to some extent the gradation, serve to provide desired surface texture.

MIXTURE DESIGN BASED ON SHEAR STRENGTH

Design Equation

The design of flexible pavements should give primary consideration to the shear strength of the various components of the pavement structure. The author has developed a design procedure based on strength theories of soil mechanics (1), which is briefly described here. The design equation is as follows:

$$\gamma z + p\left[1 - \left(\frac{1}{1 + \left(\frac{a}{z}\right)^2}\right)^{3/2}\right] = \gamma z\left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^2 + \frac{4c}{1 - \sin\phi}\left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^{1/2} \tag{3}$$

in which

p = surface contact pressure,
a = radius of equivalent circular contact area,
z = thickness of flexible pavement structure,
γ = bulk density (unit weight) of surcharge material,
ϕ = angle of internal friction of bearing material, and
c = cohesion of bearing material.

Eq. 3 is applicable to any layer of the flexible pavement structure. For instance, if $z = 0$, the equation could be applied to surface mixtures, and would be reduced to

$$p = \frac{4c}{1 - \sin\phi}\left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^{1/2} \tag{4}$$

The left side of Eq. 3 contains the familiar Boussinesq expression for the vertical intensity of stress along the axis below a circular loaded area, plus the intensity of stress due to the weight of the pavement. The right side of the equation gives the resistance which must be developed in the pavement at depth z if failure of an element from vertical stress is prevented. Lateral confinement is provided by the passive lateral pressure of a wedge of the pavement component subjected to a vertical load equal to the weight of the overlying pavement. Eq. 4 is identical to the McLeod (2) stability equation developed from the Mohr diagram for bituminous paving mixtures.

Because of the various assumptions made in the development of a design procedure based on the strength of materials, such a procedure must be substantiated by engineering practice. This should not detract from the soundness of the procedure but will help explain the reaction of materials to service conditions. Pavement loading and environmental conditions are difficult to simulate in the laboratory, and for this reason a correlation with either service performance or an established design procedure is essential.

Traffic Factor

The design equation for surface mixtures should include a traffic factor, T, to provide for variations in traffic volume and loading. The effect of this factor is to require a thicker and stronger pavement component for a heavy-duty pavement as compared with the requirement for a roadway having light to moderate traffic. The design equation for surface mixtures may then be written:

$$pT = \frac{4c}{1 - \sin\phi}\left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^{1/2} \tag{5}$$

Suggested traffic factors for design are as follows:

Highway, light traffic, short life—1.0;
Highway, moderate traffic, medium life—1.5;

Highway, heavy-duty—2.0; and
Airport taxiways, aprons, hardstands, runway ends—2.0.

INSTRUMENTATION

Two current standard testing procedures for the determination of the strength of bituminous mixtures (7) are the Marshall method (ASTM Designation: D 1559) and the Hveem method (ASTM Designation: D 1560). These are both empirical tests and because of specimen dimensions and test procedure, the data obtained cannot be subjected realistically to a theoretical analysis of stresses for purposes of pavement design. There exists at present no standard procedure for the testing of paving mixtures in flexure, tension, or triaxial compression. The standard procedure for the testing of paving mixtures in compression (ASTM Designation: D 1074-60) uses a specimen of low height-to-diameter ratio which would be unacceptable in a rational approach to the design of paving mixtures.

For a number of years there appeared in the mix design manual of the Asphalt Institute (8) the Smith triaxial method for the design of asphalt paving mixtures. The procedure is not included in the current issue of this manual. The basis for design by the Smith triaxial method (9) was the shearing resistance of the paving mixture. Correlating studies with pavements in service indicated that the procedure could be used to determine the suitability of mixtures.

For some reason the Smith triaxial method has not been widely accepted, even though desirable test specimen dimensions are used and values of the angle of internal friction and cohesion are obtained. The fact that the closed-system rather than the open-system triaxial test is used should not be a practical deterrent because fewer specimens are required in a testing program even though the data may not be theoretically correct. The primary reasons for the failure of the Smith triaxial method to receive wide acceptance seem to be specimen preparation procedure and specimen size.

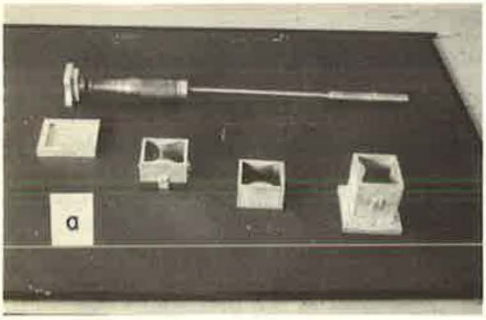

Figure 2. Compaction mold and hammer for (a) tension specimen and (b) compression and flexure specimen.

Specimen Preparation

A practical procedure, which may be applicable to both field and laboratory use, is desirable for the preparation of test specimens for flexure, tension, and triaxial compression. Triaxial specimens should have a ratio of height to minimum cross-section of 2.0. The same compaction procedure should be used for specimens for the flexure, tension and triaxial compression tests. Provision for dynamic compaction of the type used in the preparation of Marshall specimens would be desirable when considering possible field use because of portability.

The author has designed equipment providing specimens of desired dimensions which will require the same compactive effort as the standard Marshall test. All molds were designed to have the same surface area as a Marshall specimen. In this way, the same compactive effort should produce about the same density as the Marshall specimens. The tension specimen

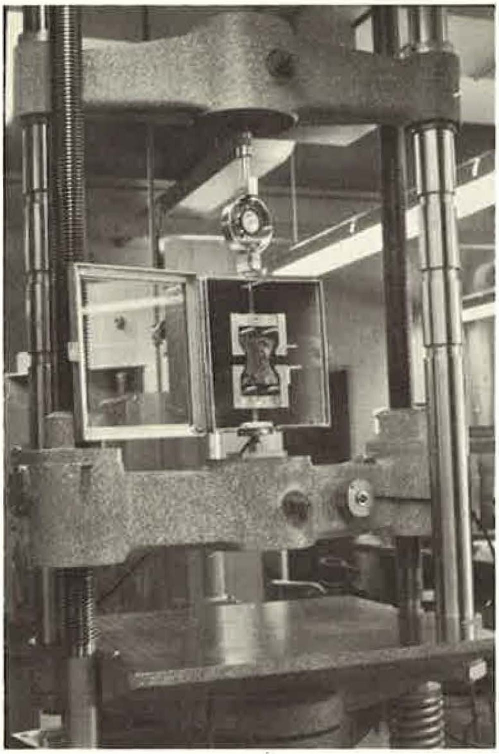

Figure 3. Tension apparatus with specimen in position for test.

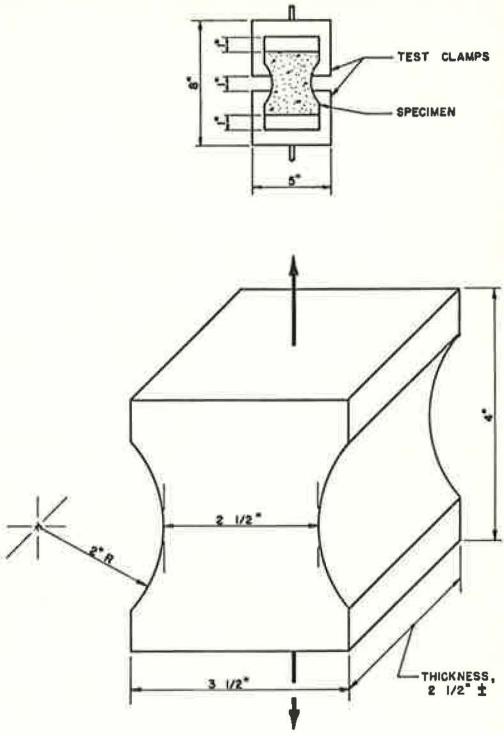

Figure 4. Principal dimensions for tension specimen and tension test clamps.

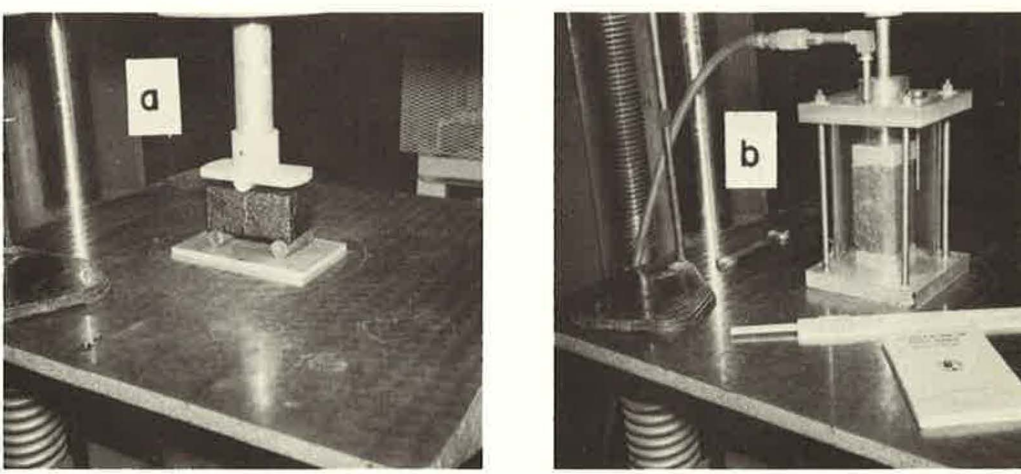

Figure 5. Specimens in position for test: (a) flexure, and (b) triaxial compression.

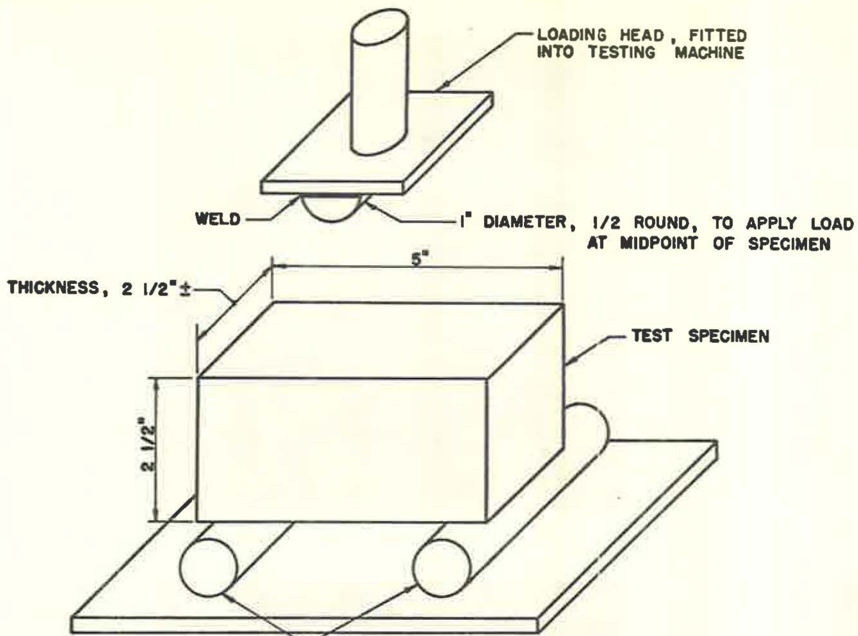

Figure 6. Flexure apparatus.

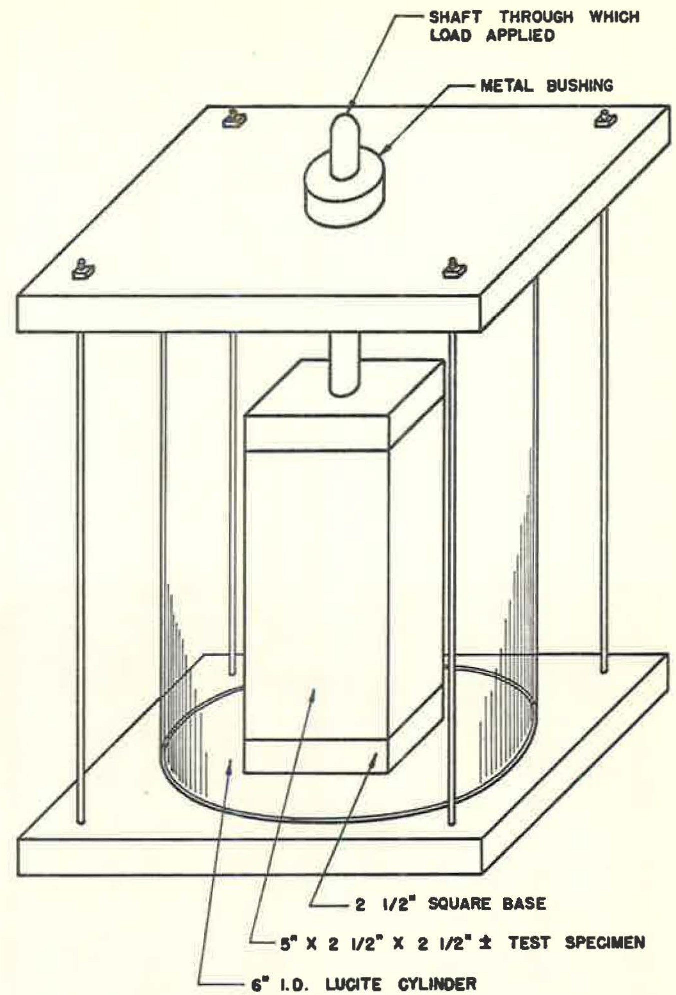

Figure 7. Triaxial cell for specimen having square cross-section.

has a minimum cross-sectional area of 2 ½ in. by the height of the specimen, which may vary slightly from 2 ½ in. Tension specimens are molded with flat top and bottom surfaces and with a shoulder at the ends for tension clamps. The unconfined compression, flexure, and triaxial specimens are made in the same molds and differ fron one another only in the manner in which they are tested. These specimens are 5 in. long, 2 ½ in. wide, and approximately 2 ½ in. high. All specimens are compacted in the same manner as the Marshall specimens—50 blows on each side of the specimen by a 10-lb weight dropping 18 in. The only differences among the compaction hammers are the shapes of the hammer ends necessitated by the different shapes of the molds. The equipment for specimen preparation is shown in Figure 2.

Both the Marshall and Hveem specimens have a minimum dimension of 2 ½ in., which is the same as the minimum dimension for flexure, tension, and triaxial compression specimens prepared in this investigation. The molds used gave satisfactory performance for mixtures having a maximum size aggregate of ½ in. They are considered adequate for mixtures having aggregate as large as ¾ in.

Strength Test Instruments

The tension apparatus used in this investigation is shown in Figures 3 and 4. The tensiometer was adapted to a Universal testing machine. This apparatus was designed with a grooved baseplate and grooved cover plates with the idea that these would be required to hold the specimen in position during the test. It was discovered, however, that the shoulder clamps were adequate. A thermostatically controlled box is provided for temperature control.

Figure 5 shows flexure and triaxial specimens in position for testing. The beam-shaped flexure specimen is tested under simple midspan bending conditions. The specimen is supported on unrestrained 1-in. diameter steel rollers spaced 4 in. apart, and the load is applied vertically through a restrained 1-in. diameter steel bar placed midway between the roller supports. The vertical dimension of the flexure specimen in test position was always 2 ½ in. for the data reported in this paper. A sketch of the flexure test apparatus is shown in Figure 6.

The triaxial cell was designed for testing of a specimen having a square cross-section, as shown in Figure 7, using compressed air for lateral confinement. A rubber membrane surrounds the test specimen and is held in place by rubber bands. An air bleeder valve is provided for drainage from the specimen during the test. The triaxial cell is set in the thermostatically controlled box during test at temperatures other than 80 F. A water bath is used for temperature control before testing.

MATERIALS

Aggregate

The limestone aggregate used in mixtures in this investigation was obtained from the Cayuga Crushed Stone Co., a source approved by the State of New York Department of Public Works for bituminous mixtures.

Aggregate was obtained from stockpiles and was oven-dried before separation into desired fractions with the Gilson machine. The fractions retained on the No. 20 sieve and larger were then washed and oven-dried before batching. This washing operation was considered essential to provide desired control on aggregate composition.

The mixtures were prepared using either gradation A or B, as shown in Figure 1. Actual values for gradation and the apparent specific gravity for each aggregate combination are given in Table 1.

Binder

A number of materials were used in various combinations and at different binder contents. The investigation was designed to show the effect of type of binder on the strength characteristics of paving mixtures. In this connection polyethylene from two sources was used as a part of the binder in some mixtures. The material most widely used, however, was 85-100 penetration grade asphalt cement.

TABLE 1

AGGREGATE PROPERTIES

Property	Value	
	Gradation A	Gradation B
Passing sieve, %:		
1/2 in.	100	100.0
1/2 in.	70	78.6
1/8 in.	55	47.1
No. 20	31	21.4
No. 80	12	7.5
No. 200	6	4.3
Apparent sp. gr.	2.723	2.708

TABLE 2

PHYSICAL PROPERTIES OF ATLANTITE 2 FLAKE ASPHALT

Property	Value
Sp. gr. at 60 F	1.06
Wt/gal	8.89
Flash pt. (C. O. C.), °F	560
Softening pt. (R & B), °F	300
Penetration:	
At 77 F, 100 g, 5 sec, 0.01 mm	2
At 150 F, 200 g, 60 sec, 0.01 mm	38
Saybolt Furol viscosity at 450 F, sec	810

TABLE 3

TENITE POLYETHYLENE 840A PHYSICAL PROPERTY DATA

Property	Value
Melt Index	20 ± 3
Density	0.923
Brittleness temp., C	-12
Tensile strength, psi:	
At fracture	1,760
At upper yield	1,840
Elongation, %	50
Thermal coef. of expansion, in./in./°F	11×10^{-5}
Melt temp., F	230
Water absorption, %	0.01

Asphalt cement was obtained at the Three Rivers plant of the Atlantic Refining Co. Flake asphalt (Atlantite 2), having physical properties as presented in Table 2, was provided by the Atlantic Refining Co. from their Philadelphia plant. Asphalt cement of 85-100 penetration was blended at 450 F with 20 percent flake asphalt (Atlantite 5) to obtain a binder having a penetration of 12.

The Allied Chemical Corp. provided A-C polyethylene 629 and A-C polyethylene 680. The 680 grade is slightly harder than the 629 grade. Some of its typical physical properties are as follows:

Softening point, 229 F;
Density, 0.94;
Acid number, 16;
Hardness, 2.

Eastman Chemical Products, Inc., supplied Tenite polyethylene which is a rather tough material of high tensile strength. A melt temperature of 230 F required elevated temperatures for mixture preparation. Typical physical property data for Tenite polyethylene 840 A are given in Table 3.

Both types of polyethylene were of low to medium molecular weight having density slightly below that for asphalt cement. A precaution was taken to prevent prolonged heating of this material at elevated temperatures.

Indopol polybutene (H-300) was provided by the Amoco Chemical Corp. and was used as a blending agent with A-C polyethylene 629. Parallel tests by the Marshall method were conducted to determine the effect of replacing 10 percent A-C polyethylene 629 with polybutene, and there was no significant difference between the Marshall stability and flow values for mixtures having binder contents of 3, 4, 5, 6, and 7, percent.

Viadon, a petrochemical binder, was provided by the Humble Oil and Refining Co. Physical properties are not included in this paper.

Asbestos fibers (7M) were used in one series of tests to show their effect on the strength characteristics of asphalt concrete. These fibers were used at a rate of application of 2 percent of the total mix with gradation B mixture which has 4.3 percent passing the No. 200 sieve.

TEST SPECIMEN PREPARATION

Binder Content

The binder content used for most of the mixtures was determined by the Marshall method of mix design. Duplicate specimens were prepared at various binder contents with the ultimate selection of binder content for various strength specimens based on air voids in total mix and stability. In most instances voids fell within the range of 3 to 5 percent.

Asphalt Concrete

In the preparation of paving mixtures using 85-100 penetration grade asphalt, both the aggregate and asphalt were heated to 300 F. Aggregate had been batched for single specimens and was allowed to remain in an electric oven for a minimum of 12 hr before mixing. The asphalt was brought to temperature in an electric pot from which it was weighed directly into a mixing bowl containing the aggregate for a single specimen. The material was mixed with an electric mixer for 1 min. The mix was then placed in a heated mold, rodded 25 times, and compacted with a dynamic hand-operated compactor. The molds were placed under running water to cool before extrusion. Water was permitted to fall directly on the mold rather than on the specimen to reduce water absorption into the specimen before dry weight for density determination could be made. In most instances, specimens were tested the day after they had been prepared.

The temperature of both aggregate and asphalt was raised to 350 F for mixtures using asphalt cement having a penetration of 12.

Asphalt Cement—Flake Asphalt Binder

Mixtures in which flake asphalt was included in the binder with 85-100 penetration grade were prepared by heating the aggregate to 450 F, adding the flake asphalt in dry form, and mixing until the asphalt was melted and distributed. The desired amount of penetration grade asphalt was then weighed in at 300 F followed by 1 min of mechanical mixing.

A-C Polyethylene

In the preparation of mixtures containing A-C polyethylene 629 or 680 blended with 85-100 penetration grade asphalt, both the stone and asphalt were heated to 300 F, the desired amount of asphalt was added to the aggregate followed by mechanical mixing to obtain uniform distribution, and the polyethylene was added in dry form with additional mechanical mixing.

For the mixture having a binder blend of A-C polyethylene 629, Indopol polybutene H-300, flake asphalt, and 85-100 penetration grade asphalt, the asphalt cement was heated to 450 F and the aggregate to 550 F. Flake asphalt was added first to the heated aggregate, followed by penetration grade asphalt and mechanical mixing. After blending and distribution of the asphalt binder with the aggregate, the A-C polyethylene 629 was added in dry form with continued mixing.

Tenite Polyethylene

Two series of test specimens were made using a blend of Tenite polyethylene and 85-100 penetration grade asphalt. The binder was prepared by heating the asphalt to 500 F and blending Tenite with the asphalt in an asphalt pot. The blend was held at this temperature for application to the stone, also heated to 500 F. The material was then mixed mechanically until thorough distribution was obtained. Compaction temperatures were normally about 350 F.

Mixtures having a blend of flake asphalt (Atlantite 2) and Tenite as binder were prepared by first applying the dry asphalt to stone at 550 F, followed by mechanical mixing, the addition of dry polyethylene, and continued mixing.

Viadon

The procedure recommended by the Humble Oil and Refining Co. for the preparation of laboratory specimens using Viadon binder was followed. This consisted of heating the total aggregate to 425 ± 10 F and placing it in a heated mixing bowl. The dry resin was added while mixing, followed by the addition of the plasticizer and colored paste. The colored paste was weighed on a small piece of polyethylene film, and the film and paste were added to the mix.

TABLE 4

INFLUENCE OF TEST TEMPERATURE ON STRENGTH OF ASPHALT CONCRETE[a]

Temperature (°F)	Flexure (MR) (psi)	Tension (psi)	Triaxial Compression (psi)		
			0-Psi Lat.	15-Psi Lat.	30-Psi Lat.
40	526	120	426	511	641
80	71	14	107	169	221
110	25	6	63	118	179
140	14	1.5	33	83	174

[a]Average of duplicate specimens using gradation A, 6 percent asphalt (85-100 penetration), 0.05-in./min loading.

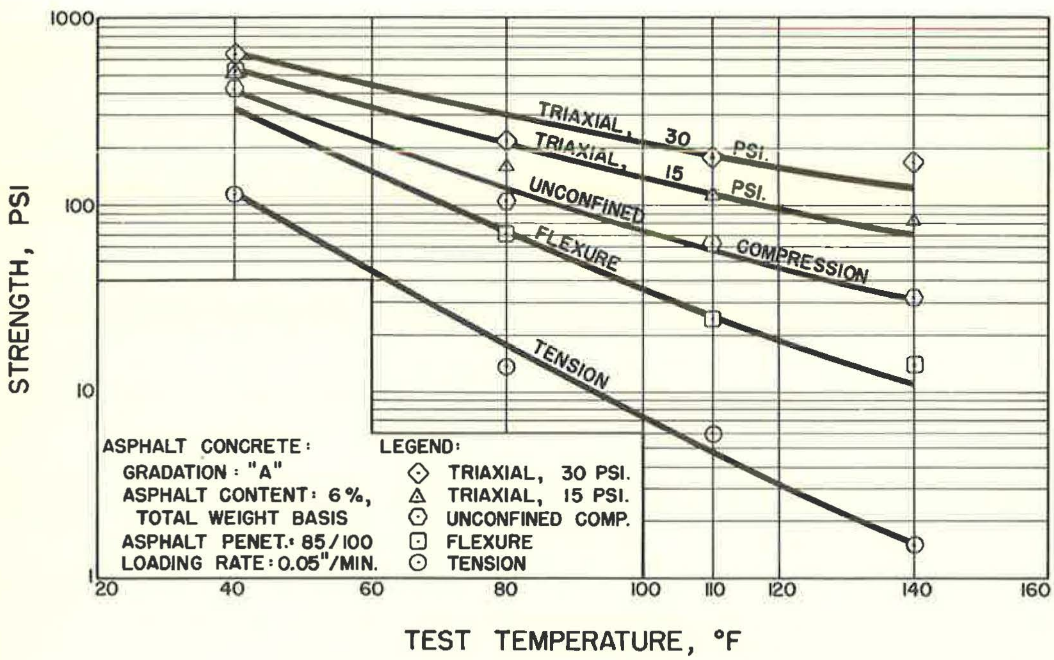

Figure 8. Influence of test temperature on strength of asphalt concrete.

Asbestos Fibers in Mixture

Both the stone and 85-100 penetration grade asphalt were heated to 300 F. The asbestos fibers were added to the dry stone and dry mixed as would be the case in the addition of a mineral filler at the plant. The asphalt was then added with mechanical mixing.

TEST RESULTS AND DISCUSSION

Test Temperature

The influence of test temperature on the strength of asphalt concrete with aggregate gradation A and 6 percent 85-100 penetration grade asphalt, tested at a rate of loading of 0.05 in./min, is indicated in Table 4 and Figure 8. Flexure, tension, unconfined compression and triaxial tests were run. The significant observation from this series of tests is the tremendous influence of temperature on the tensile strength of asphalt concrete. As the temperature is lowered from 140 to 40 F, the tensile strength increases more than 50 times. Lateral confinement of a test specimen tends to reduce the effect of temperature on shear strength.

The Mohr diagram using the unconfined compression and triaxial data was drawn on sheets of 11- by 17-in. graph paper from which the angle of internal friction and cohesion were obtained. A part of this investigation was a determination of whether or not the tension test along with the unconfined compression test could be used to determine the shear strength parameters of friction and cohesion. In Figure 9 the dashed line is made tangent to the tension and unconfined compression curves, giving a visual indication of the difference which may be expected by the two methods. The influence of test temperature on the angle of internal friction and cohesion is indicated in Figure 10 and Table 5. Cohesion increases with a decrease in temperature; however, the angle of internal friction as determined in this test decreases with a decrease in temperature from 140 F to about 95 F, at which point the friction angle as

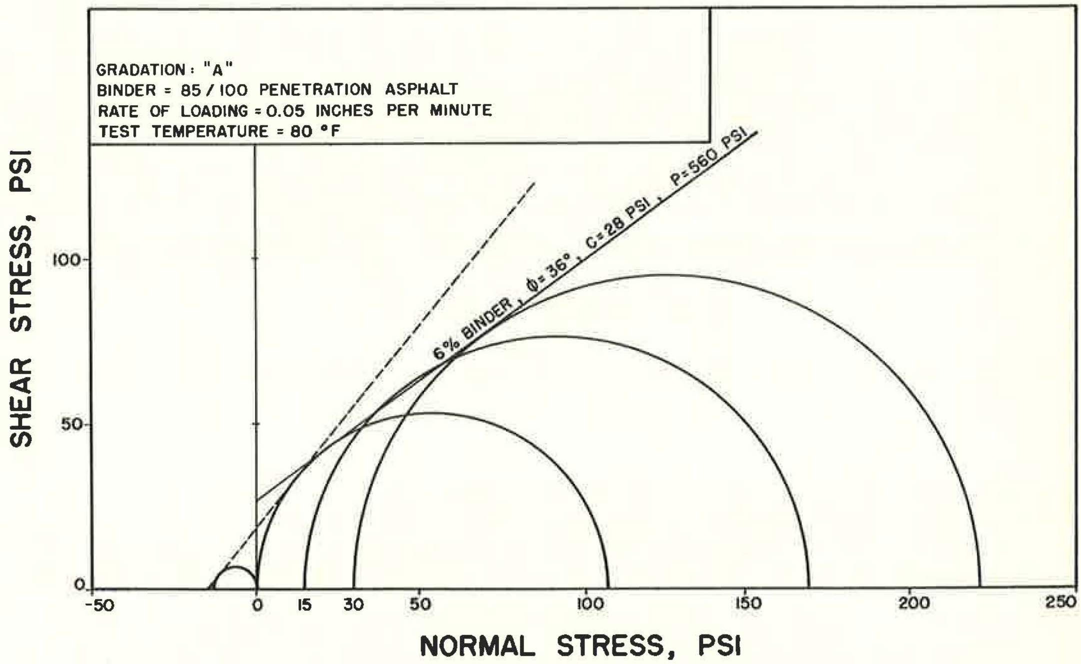

Figure 9. Mohr diagram for asphalt concrete; angle of internal friction and cohesion determined from triaxial compression circles.

TABLE 5

SHEAR STRENGTH PROPERTIES OF ASPHALT CONCRETE[a]

Temperature (° F)	Rate of Load (in./min)	Triaxial Data			Tension-Comp. Data		
		ϕ (deg)	c (psi)	p^b (psi)	ϕ (deg)	c (psi)	p^b (psi)
140	0.002	43	1	29	-	-	-
140	0.05	40	6	145	61	3	372
140	0.5	39	14	310	53	10	600
140	2.0	38	18	390	50	14	653
40	0.05	50	75	3,500	34	115	2,070
80	0.05	36	28	560	51	20	512
110	0.05	36	16	300	55	10	706

[a]Based on duplicate specimens of gradation A, 6 percent asphalt (85-100 penetration).
[b]From Eq. 4.

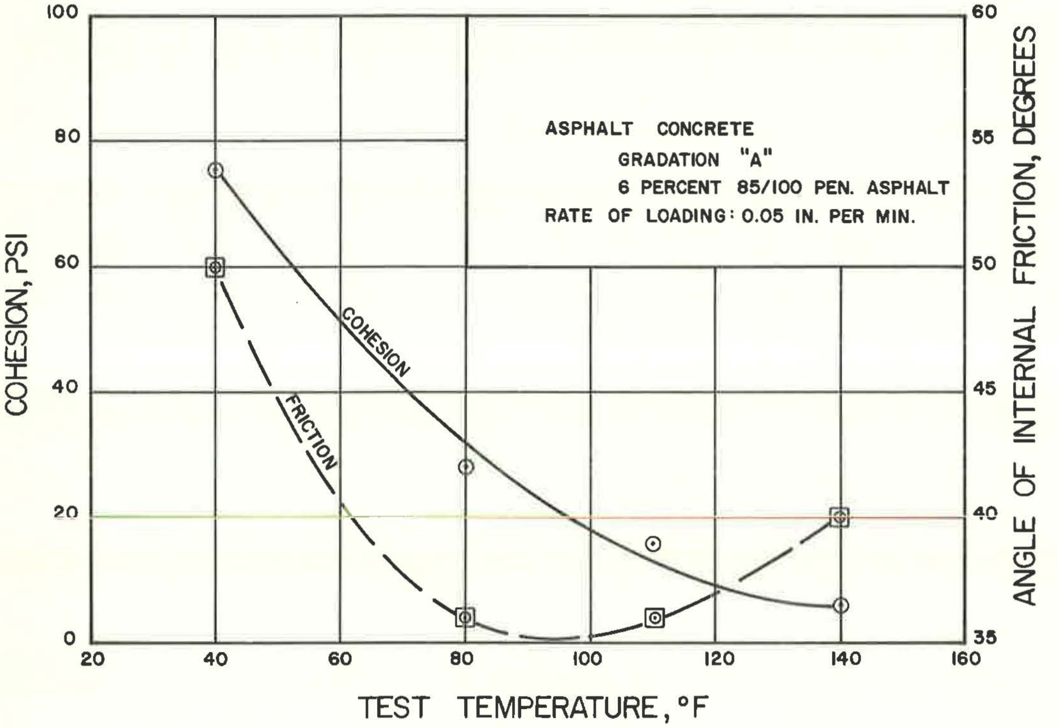

Figure 10. Influence of test temperature on angle of internal friction and cohesion for asphalt concrete.

determined from the Mohr diagram increases rapidly with a further decrease in temperature. Doyle (10) states that, at low temperatures, asphalt reacts somewhat as a solid, so this may be the explanation for the friction curve as shown in Figure 10.

The values of the angle of internal friction and cohesion as shown in Figure 10 were used to compute pavement resistance to vertical contact pressure using Eq. 4. When plotted on a log-log graph (Fig. 11), the relationship between pavement resistance and temperature gives a straight line. In making a determination of the influence of temperature on pavement resistance for purposes of design, it is found that pavement resistance as computed by Eq. 4 is four times greater at 80 F than at 140 F.

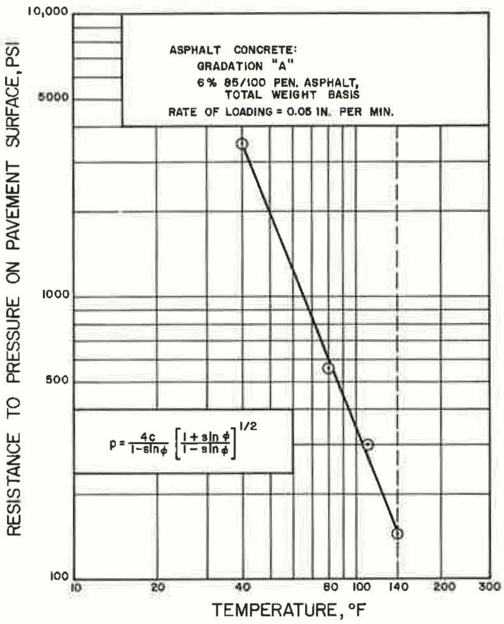

Figure 11. Influence of temperature on pavement resistance to vertical contact pressure.

Rate of Loading

The influence of rate of loading on strength of asphalt concrete is indicated in Table 6 and in Figure 12. It is again apparent that the tensile strength of asphalt concrete is very sensitive to test conditions. The tensile strength at 2.0 in./min is more than 50 times the tensile strength at 0.002 in./min. Lateral confinement of test specimen tends to reduce the effect of rate of loading on strength.

The influence of rate of loading on the angle of internal friction and cohesion for asphalt concrete tested at 140 F is indicated in Table 5 and Figure 13. Also given in Table 5 are values of the angle of internal friction and of cohesion as determined from the tension-unconfined compression data at various test temperatures and rates of loading. It is significant that there is not a consistent relationship between these data and those determined from triaxial compression tests. It was concluded that triaxial tests should be used for the determination of shear strength data.

The influence of rate of loading on pavement resistance to vertical contact pressure as computed using Eq. 4 is indicated in Table 5 and Figure 14.

Table 7 presents data on the influence of test temperature and rate of loading on the flexural strength and magnitude of deflection at failure for asphalt concrete. These data show that the magnitude of deflection at failure increases with an increase in rate of loading and with a decrease in test temperature. The influence of rate of loading on deflection for the flexure test is shown in Figure 15.

Asphalt Content

The influence of asphalt content on unconfined and triaxial compression for asphalt concrete having aggregate gradation B and tested at 80 F and 0.05-in./min rate of loading is shown in Table 8 and in Figure 16. The Mohr diagram for a binder content of 6 percent is shown in Figure 17. This may be compared with the Mohr diagram

TABLE 6

INFLUENCE OF RATE OF LOADING ON STRENGTH OF ASPHALT CONCRETE[a]

Loading Rate (in./min)	Flexure (MR) (psi)	Tension (psi)	Triaxial Compression (psi)		
			0-Psi Lat.	15-Psi Lat.	30-Psi Lat.
0.002	1.6	0.16	4.2	-	-
0.05	14	1.5	33	83	174
0.5	13	6.8	55	94	181
2.0	30	9.8	74	131	199

[a]Average of duplicate specimens of gradation A, 6 percent asphalt (85-100 penetration), 140 F.

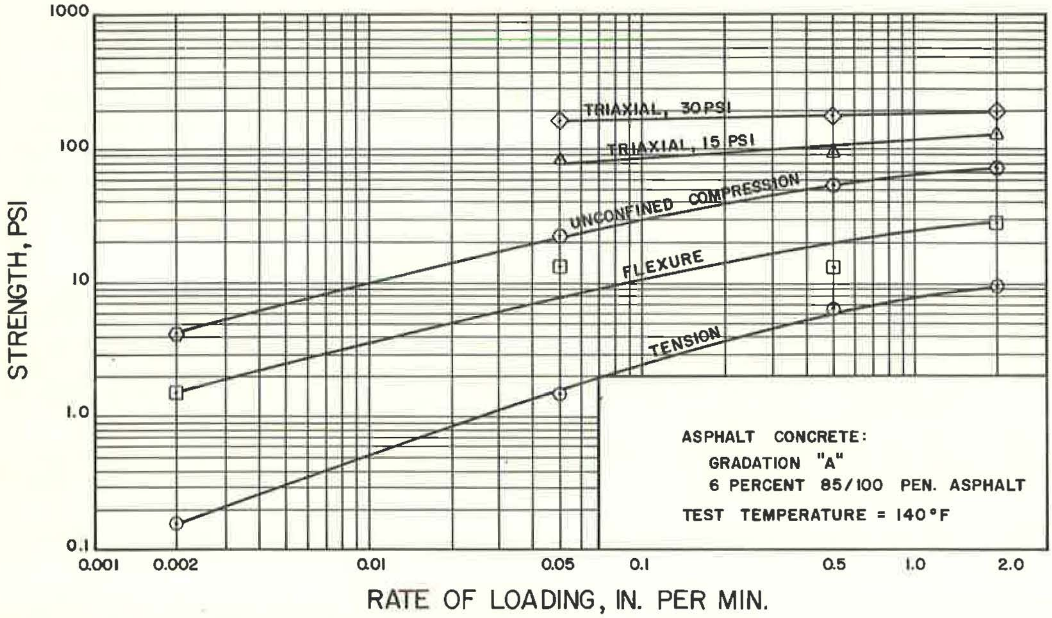

Figure 12. Influence of rate of loading on strength of asphalt concrete.

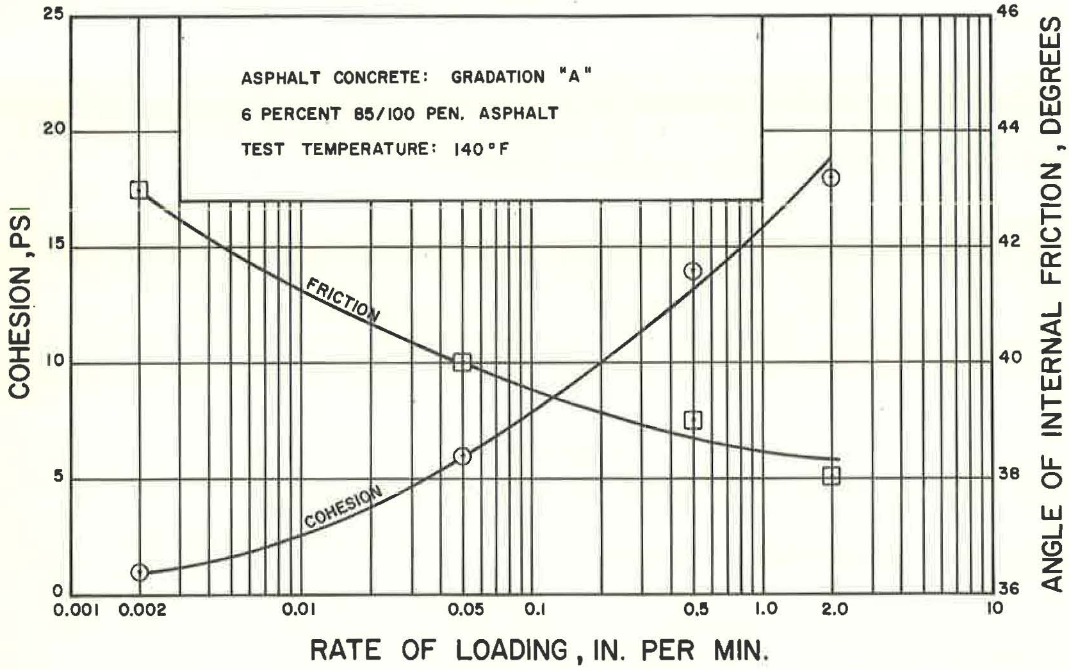

Figure 13. Influence of rate of loading on angle of internal friction and cohesion for asphalt concrete.

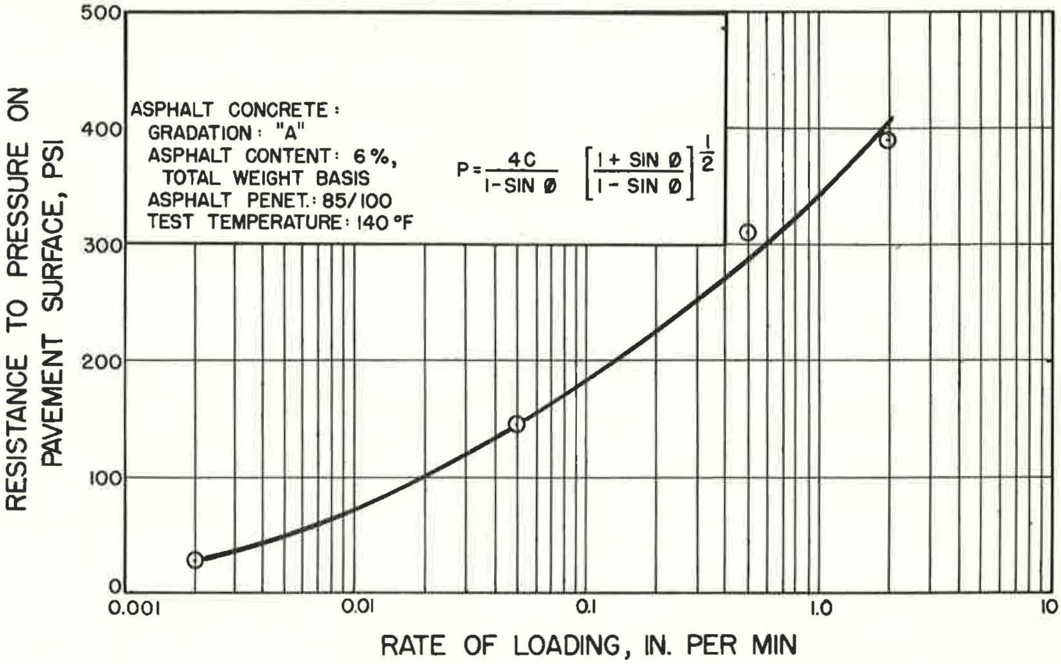

Figure 14. Influence of rate of loading on pavement resistance to vertical contact pressure.

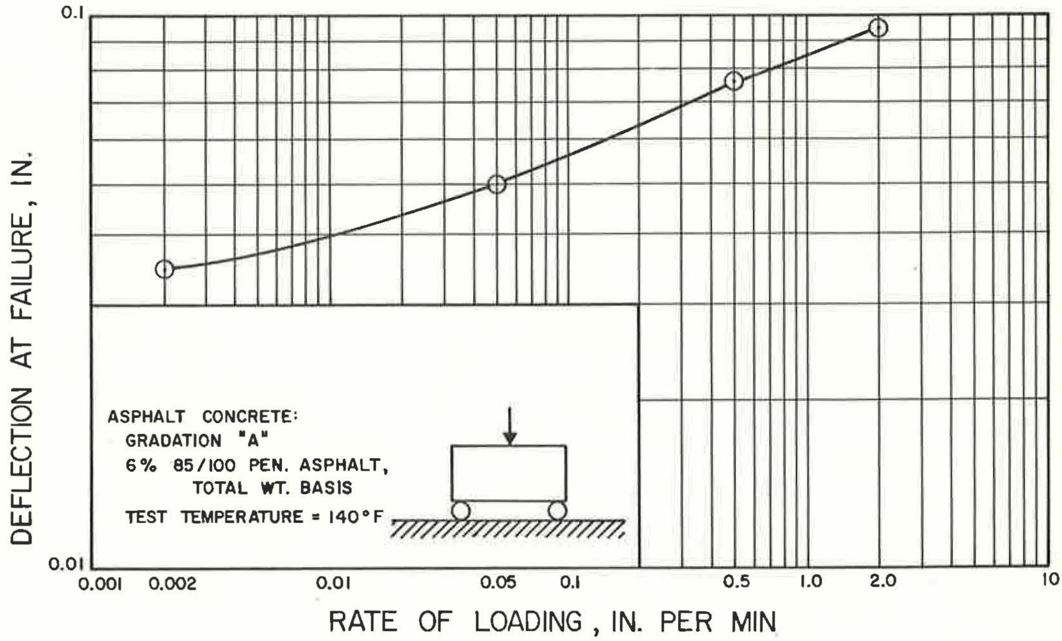

Figure 15. Influence of rate of loading on flexural deflection at failure for asphalt concrete.

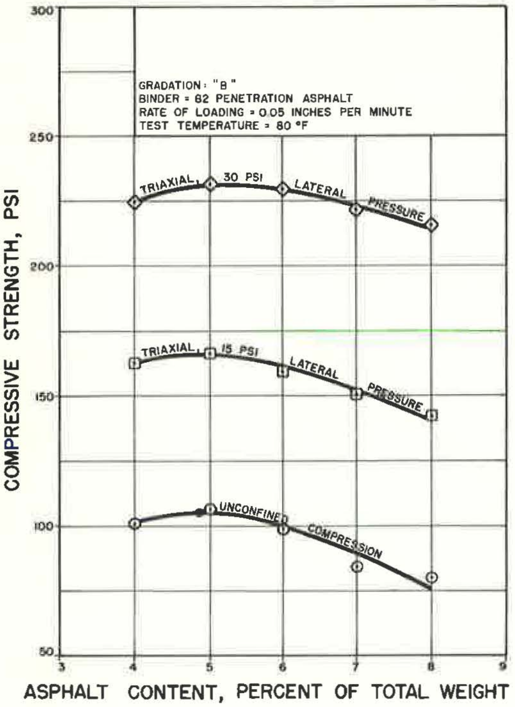

Figure 16. Influence of asphalt content on strength of asphalt concrete in triaxial compression.

presented in Figure 9 for the same conditions of test in which aggregate gradation A was used. Gradation A, which has more fines and is a slightly tighter mix, produces slightly higher cohesion but a lower angle of internal friction than gradation B. The influence of asphalt content on angle of internal friction, cohesion, and pavement resistance to vertical content pressure is shown in Figure 18. Maximum cohesion occurs at 4.5 percent asphalt, whereas the maximum angle of internal friction occurs at 6.5 percent asphalt, and as the

TABLE 7

INFLUENCE OF TEMPERATURE AND RATE OF LOADING ON FLEXURAL STRENGTH AND DEFLECTION[a]

Temperature (°F)	Rate of Load (in./min)	Mod. of Rupture (psi)	Deflection (in.)
140	0.002	1.6	0.035
140	0.05	14	0.050
140	0.5	13	0.076
140	2.0	30	0.095
110	0.05	25	0.080
80	0.05	71	0.089
40	0.05	526	0.105

[a]Average of duplicate specimens for gradation A, 6 percent asphalt (85-100 penetration).

TABLE 8

INFLUENCE OF ASPHALT CONTENT ON STRENGTH OF ASPHALT CONCRETE[a]

Asphalt[b] (%)	Unconf. Comp. (psi)	Triaxial Comp. (psi)		ϕ (deg)	c (psi)	Pave. Resist. (psi)
		15-Psi Lat.	30-Psi Lat.			
4	102	163	224	37.5	25	508
5	107	167	232	39	25	565
6	98	161	230	40	23	553
7	84	151	221	40	20	482
8	80	142	216	38.5	19	418

[a]Average of triplicate specimens for gradation B, 0.05-in./min rate of loading, 80 F.
[b]82 penetration.

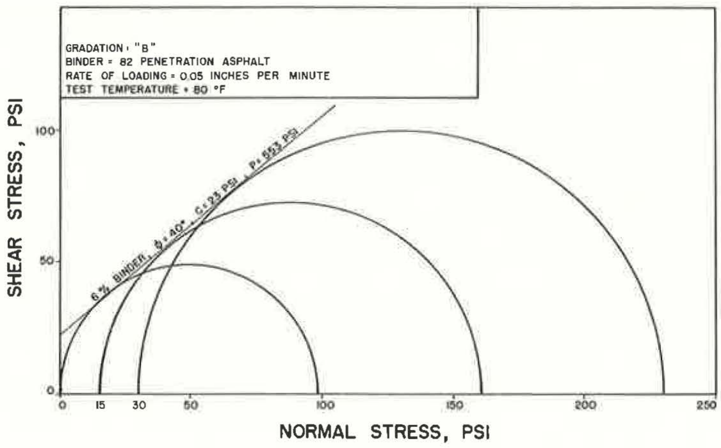

Figure 17. Mohr diagram for asphalt concrete.

asphalt content increases beyond this point the effect of lubrication is pronounced as friction drops rapidly. The effect of film thickness on cohesion at this test temperature is apparent, showing that thick films of asphalt have less resistance to internal shear than thin films. The curve for pavement resistance is similar to curves developed by the standard Marshall method of test and has maximum resistance at an asphalt content of about 5.5 percent.

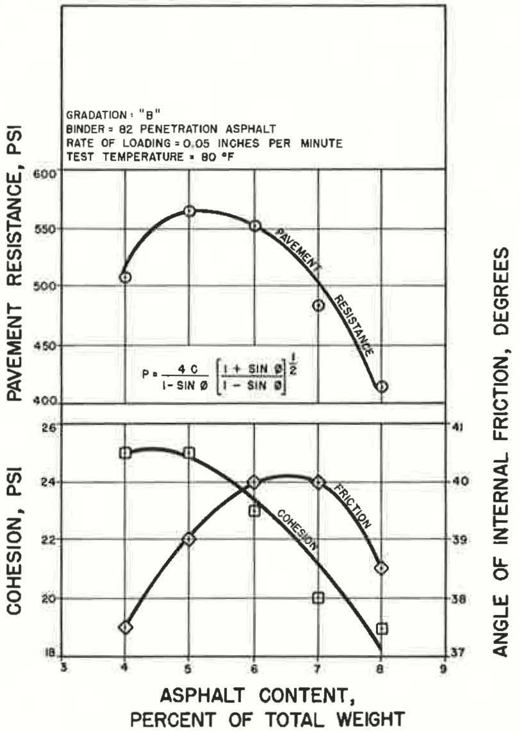

Figure 18. Influence of asphalt content on angle of internal friction, cohesion, and pavement resistance to vertical contact pressure for asphalt concrete.

Figure 19 shows the Mohr diagram for a mixture having 5.75 percent asphalt, with the same mix composition as those mixtures with properties given in Figure 18. The difference in test temperature of 140 F shows the much lower value for cohesion and a higher value for the angle

TABLE 9

STRENGTH OF ASPHALT CONCRETE AT OPTIMUM BINDER CONTENT FOR GRADATION B AGGREGATE[a]

Property	Strength
Marshall stability, lb[b]	1,740
Marshall flow value	12.5
Tension, psi	0.15
Unconfined compression, psi	19
Triaxial, 15-psi lateral, psi	119
ϕ, deg	47.5
c, psi	4.0
Pavement resistance, psi	157

[a]Average of duplicate specimens, 5.75 percent asphalt (85-100 penetration), 0.05-in./min rate of loading, 140 F.
[b]Marshall test at 2.0 in./min.

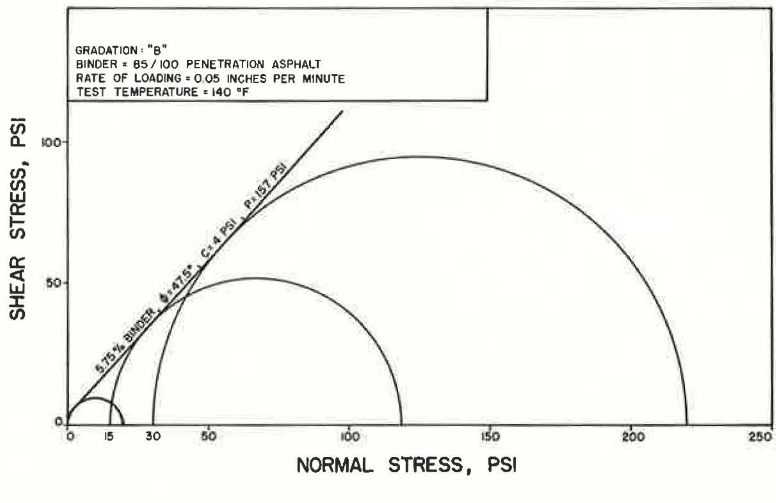

Figure 19. Mohr diagram for asphalt concrete.

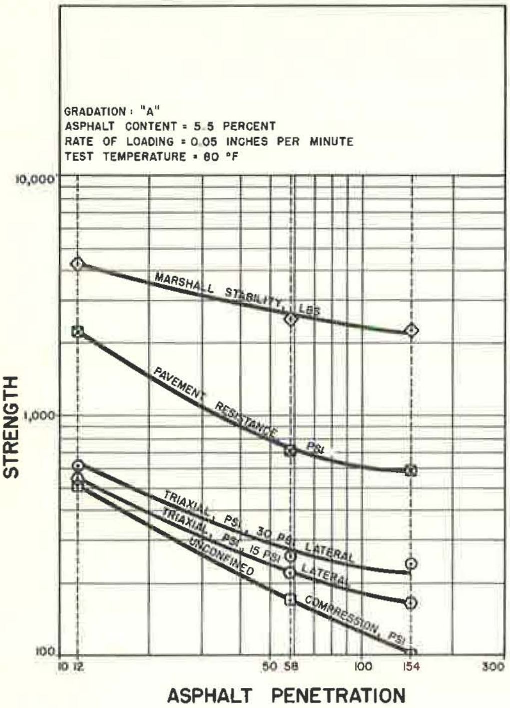

Figure 20. Influence of asphalt penetration on strength of asphalt concrete; Marshall test at 140 F and 2.0 in./min.

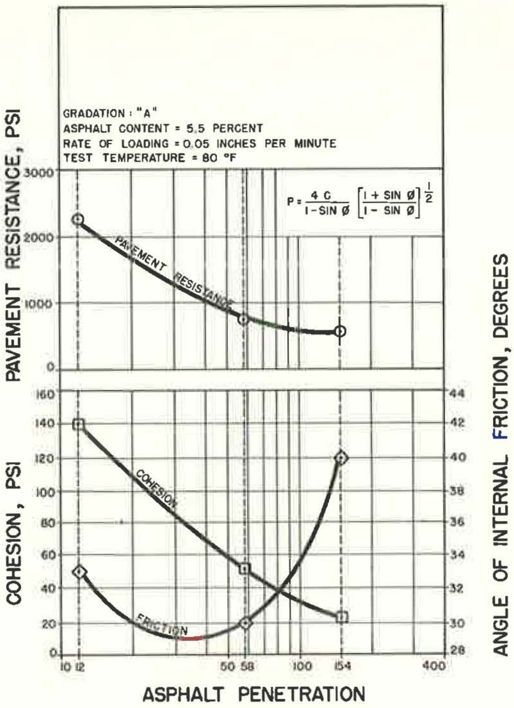

Figure 21. Influence of asphalt penetration on angle of internal friction, cohesion, and pavement resistance to vertical contact pressure.

TABLE 10

INFLUENCE OF ASPHALT PENETRATION AND VISCOSITY ON STRENGTH OF ASPHALT CONCRETE[a]

Property	Strength		
	12 Pen.	58 Pen.	154 Pen.
Viscosity, 10^5 poises	2,000[b]	45.5	7.6
Marshall stability, lb[c]	4,360	2,595	2,375
Marshall flow, 0.01 in.	23	13	24
Flexure (MR), psi	487	161	50
Unconfined compression, psi	514	177	100
Triaxial compression, psi:			
15-psi lateral	547	228	172
30-psi lateral	617	264	240
ϕ, deg	33	30	40
c, psi	140	52	24
Pavement resistance, psi	2,270	720	580

[a]Average of duplicate specimens for gradation A, 5.5 percent asphalt, 0.05-in./min rate of loading, 80 F.
[b]Estimated.
[c]Marshall test at 140 F and 2.0 in./min.

of internal friction. Test values are given in Table 9.

Asphalt Penetration and Viscosity

The influence of asphalt penetration on the strength of asphalt concrete is shown in Figure 20. The standard Marshall test was performed at 140 F and 2.0-in./min rate of loading with unconfined and triaxial compression tests run at 80 F and 0.05 in./min. Figure 21 shows the influence of asphalt penetration on angle of internal friction, cohesion, and pavement resistance to vertical contact pressure. The curve for friction follows the trend developed in the series of tests which showed the influence of test temperature on strength properties (Fig. 10). This shows that at 80 F, asphalt having a penetration below 30 begins to show properties of a solid. The apparent close correlation between Marshall stability and pavement resistance determined from triaxial test data is evident in Figure 20.

It was intended in the design of this experiment that the relationship between absolute viscosity as determined by the sliding plate microviscometer and strength properties be developed. At 77 F, however, it was discovered that the microviscometer did not have the load capacity for the 12 penetration asphalt. Table 10 gives data to show the influence of asphalt penetration and absolute viscosity on the strength of asphalt concrete.

TABLE 11

STRENGTH PROPERTIES OF ASPHALT CEMENT-FLAKE ASPHALT MIXTURES[a]

Flake (%)	Triaxial		p^b (psi)	Marshall	
	ϕ (deg)	c (psi)		Stability (lb)	Flow (0.01 in.)
0	43.5	6.5	195	—	—
5	42.0	7.0	191	2,680	17
10	44.5	7.0	222	2,852	17
15	42.0	7.5	202	2,670	16
20	43.5	7.5	224	3,325	13
25	44.5	8.0	254	3,222	14
30	—	—	—	3,627	12

[a]Based on duplicated specimens for gradation A, 5.5 percent asphalt (85-100 penetration), 0.05-in./min rate of loading, 140 F.
[b]From Eq. 4.

Flake Asphalt—Asphalt Cement Binder

The viscosity of an asphalt binder can be increased at the asphalt mix plant by adding dry asphalt to the pug mill through the mineral filler attachment. This has been done to a limited extent. In this investigation the total binder content remained constant at 5.5 percent and the amount of flake asphalt was increased from 0 to 30 percent of the binder. The influence of flake asphalt in the binder on the strength of mixture is shown in Figure 22 with the influence on the angle of internal friction, cohesion, and pavement resistance given in Table 11. These data show a slight increase in strength with an increase in flake asphalt. It is possible that there was not complete blending of the flake asphalt with the asphalt cement during the

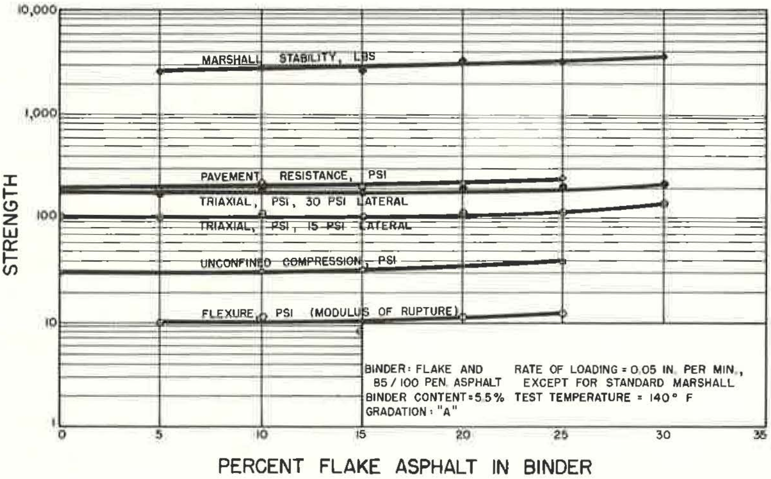

Figure 22. Influence of flake asphalt in binder on strength of asphalt concrete.

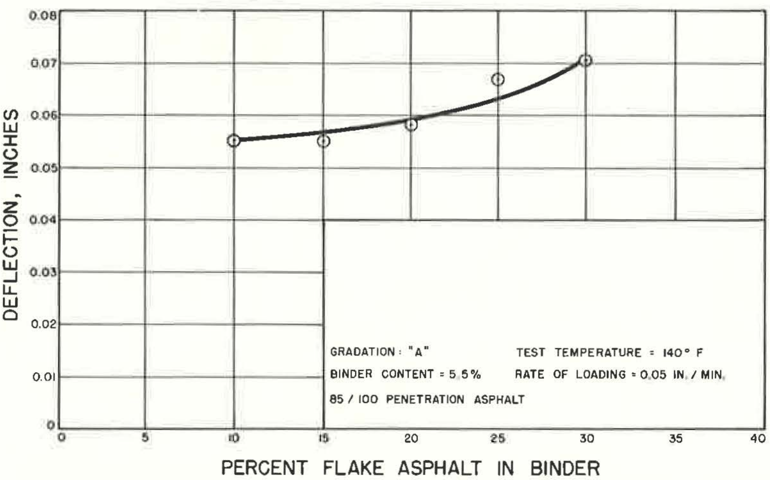

Figure 23. Influence of flake asphalt in binder on flexural deflection at failure for asphalt concrete.

laboratory mixing operation. On the basis of the data relating asphalt penetration and viscosity to the strength of asphalt concrete, it would appear that the effect of flake asphalt would be more pronounced.

Figure 23 shows that the addition of flake asphalt to the binder increases deflection at failure in the flexure test.

TABLE 12

STRENGTH OF MIXTURE HAVING BLEND OF ASPHALT CEMENT, FLAKE ASPHALT AND A-C POLYETHYLENE 629 AS BINDER[a]

Property	Strength
Flexure (MR), psi	60
Tension, psi	8.3
Unconfined compression, psi	119
Triaxial compression, psi:	
15-psi lateral	175
30-psi lateral	245
ϕ, deg	39
c, psi	29
Pavement resistance, psi	660
Marshall stability, lb[b]	3,600
Marshall flow value	11

[a]Average of duplicate specimens for gradation A, 5.5 percent binder (18 percent A-C polyethylene 629, 2 percent Indopol polybutene (H-300), 15 percent flake asphalt, and 65 percent 85-100 pen. asphalt), 0.05-in./min rate of loading, 140 F.
[b]Marshall test at 2.0 in./min.

Polyethylene in Binder

An investigation was made to determine the feasibility of using polyethylene in the binder of paving mixtures. A-C polyethylene 629 blended with polybutene, flake asphalt, and 85-100 penetration grade asphalt provided a binder which gave desirable strength characteristics to the mixture. Pavement resistance at 140 F was high as a result of high cohesion in the mixture. Strength data are given in Table 12. The influence of test temperature on the flexural strength of this mixture is shown in Figure 24.

The Marshall test was used to determine the influence of the amount of A-C polyethylene 629 blended with 85-100 penetration asphalt on the strength of mixtures. The results of this series are given in Table 13. The addition of A-C polyethylene 629 did permit a lower binder content for the mixtures.

The influence of A-C polyethylene 680 blended with 85-100 penetration asphalt on the strength of a mixture is indicated in Table 14. Marshall, unconfined compression and triaxial compression tests

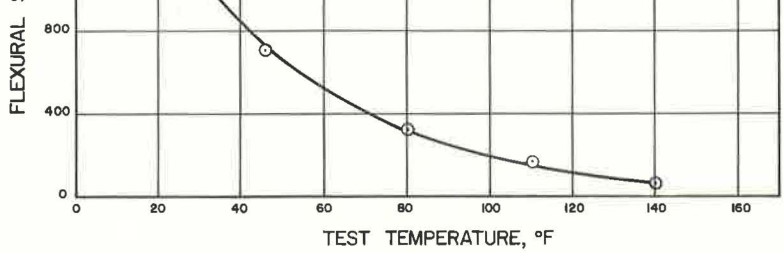

Figure 24. Influence of test temperature on modulus of rupture for mixture having binder composition of A-C polyethylene 629, Indopol, flake asphalt and penetration grade asphalt.

TABLE 13

STANDARD MARSHALL STABILITY FOR MIXTURES HAVING A-C POLYETHYLENE 629 IN BINDER WITH 85-100 PENETRATION ASPHALT[a]

Polyethylene in Binder (%)	Binder (%)	Marshall Stability (lb)
0	6.0	2,075
20	4.0	2,455
50	4.0	3,250
100	4.0	5,400

[a]Average of triplicate specimens.

TABLE 14

STRENGTH OF MIXTURE HAVING BLEND OF ASPHALT CEMENT AND A-C POLYETHYLENE 680 AS BINDER[a]

Property	Strength
Marshall stability, lb[b]	2,510
Marshall flow, 0.01 in.	9
Flexure (MR), psi	135
Unconfined compression	168
Triaxial compression, psi:	
15-psi lateral	212
30-psi lateral	250
ϕ, deg	28
c, psi	51
Pavement resistance, psi	640

[a]Average of triplicate specimens for gradation B, 5 percent binder (80 percent 85-100 penetration asphalt, 20 percent A-C polyethylene 680), 0.05-in./min rate of loading, 80 F.
[b]Marshall test at 140 F and 2.0 in./min.

TABLE 15

STRENGTH AT OPTIMUM BINDER CONTENT OF MIXTURES HAVING ASPHALT CEMENT-TENITE POLYETHYLENE BINDER[a]

Property	Strength	
	20% Tenite	35% Tenite
Binder content, %	6.0	7.0
Flexure (MR), psi	72	86
Tension, psi	21	--
Unconfined compression, psi	94	162
Marshall stability, lb[b]	3,590	3,975
Marshall flow, 0.01 in.	9	8.5

[a]Average of duplicate specimens for gradation A, 85-100 penetration asphalt, 0.05-in./min loading, 140 F.
[b]Marshall test at 2.0 in./min.

were run. A comparison with Tables 8 and 9 shows that 20 percent A-C polyethylene 680 in the binder tested at 80 F and 0.05 in./min produces an increase in the strength of the mixture of only about 15 percent. However, in testing by the Marshall method at 140 F and 2.0 in./min, there is an increase in strength of about 50 percent with the addition of 20 percent A-C polyethylene 680. The significance of this is that A-C polyethylene 629 or 680 could be used to advantage where it was desirable to increase strength characteristics at high temperatures.

Tenite polyethylene was used in mixtures in two ways. In one case a previously blended binder of Tenite and asphalt cement was used in the preparation of mixtures for the Marshall, flexure, tension, unconfined compression, and triaxial tests. The results of these tests are given in Table 15. In the second series of tests, Tenite polyethylene and flake asphalt were both added in dry form to the hot stone to provide the binder. The asphalt was added first. Various combinations of the binder were used, with the results presented in Table 16 and Figure 25. This binder was very strong as is shown by a modulus of rupture of 592 psi at 140 F where the binder consists of 80 percent Tenite. It produces a relatively rigid mixture which has low flexural deflections at failure. This was also observed when there was not a great difference in strength between the unconfined and triaxial compression tests.

Viadon

Mixtures were prepared using 5.5 and 7.4 percent Viadon as binder. The effect of binder content appears to be influenced by rate of loading as is indicated in Table 17. All tests were at 140 F. For the Marshall test which has a rate of loading of 2.0 in./min, the stability was greater at a binder content of 7.4 percent. However, on the basis of triaxial compression data as shown in Figure 26 in which the rate of loading was 0.05 in./min, the strength at 5.5 percent binder was more than twice that at 7.4 percent binder.

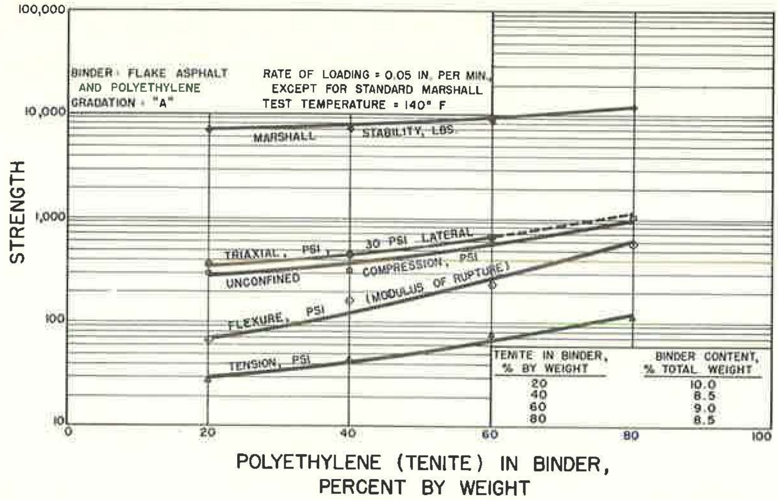

Figure 25. Influence of binder composition on strength of flake asphalt-tenite polyethylene mixtures.

TABLE 16

STRENGTH AT OPTIMUM BINDER CONTENT OF MIXTURES HAVING FLAKE ASPHALT-TFNITE POLYETHYLENE BINDER[a]

Property	Strength			
	20% Tenite	40% Tenite	60% Tenite	80% Tenite
Binder content, %	10.0	8.5	9.0	8.5
Flexure (MR), psi	66	165	240	592
Tension, psi	29	45	76	121
Unconfined compression, psi	299	325	-	1,096
Triaxial, 30-psi lateral, psi	366	453	665	-
Marshall stability, lb[b]	6,865	7,035	8,860	13,200
Marshall flow, 0.01 in.	13	10.5	10	9.5

[a]Average of duplicate specimens for gradation A, 0.05-in./min rate of loading, 140 F.
[b]Marshall test at 2.0 in./min.

TABLE 17

STRENGTH OF VIADON[a]

Property	Strength	
	5.5% Binder	7.4% Binder
Marshall stability, lb[b]	3,185	3,690
Marshall flow, 0.01 in.	18	35
Tension, psi	--	8
Unconfined compression, psi	99	50
Triaxial compression, psi:		
15-psi lateral	159	101
30-psi lateral	218	148
ϕ, deg	37	32.5
c, psi	25	14
Pavement resistance, psi	505	220

[a]Average of triplicate specimens for gradation A, 0.05-in./min rate of loading, 140 F.
[b]Marshall test at 2.0 in./min.

Asbestos Fibers in Binder

Asbestos fibers were added to a mixture using aggregate gradation B at a rate of 2 percent of the weight of the mixture. Test results are presented in Table 18. A comparison with Tables 8, 9, and 10 indicates that the addition of 2 percent asbestos fibers has about the same effect on strength as a 2 percent increase in the fines content in the aggregate. Cohesion is increased but there is some reduction in the angle of internal friction. The triaxial data show a greater effect of asbestos fibers on the strength of the mixture than do the Marshall stability data. It is possible that this strength variation with test procedure is also related to the rate of load application.

DESIGN APPLICATION

The design of flexible paving surfaces must provide shearing resistance in the pavement to excessive compression deformation under critical service conditions. The most critical condition of the pavement, with respect to shearing resistance, occurs when the pavement is at the maximum temperature and loads on the pavement are moving at a slow rate. A realistic design temperature is considered to be 140 F. The selection of a rate of load application to simulate actual loading of the pavement in service is not as easy. Although static loading would be most critical, as in the case of parked vehicles, this would not be practical for use in the design of pavements for highways, airport runways or streets. Also, it would not be practical to design a pavement for the shear stresses which would develop for vehicles moving at the design speed of the roadway. Another factor to be considered is the magnitude of compressive deformation in the surface and the relationship of this deformation to the total deformation of a pavement surface with loading, a part of which would be due to flexure resulting from the total deformation of the flexible pavement structure. A rate of loading of 0.05 in./min would be realistic for laboratory testing of specimens in unconfined and triaxial compression. Data obtained under these conditions of temperature and loading should be evaluated on the basis of pavement performance to determine a correlation.

On the basis of data obtained in this investigation of asphalt concrete and as provided in Tables 4 through 6 and Figures 8 and 10 through 14, mixtures tested at a rate of loading of 0.05 in./min are 4 times as strong at 80 F as at 140 F. Also, specimens tested at 140 F are twice as strong when tested at a rate of loading of 0.5 in./min as when tested at 0.05 in./min. These and other established relationships developed for asphalt concrete were used to convert computed values for pavement resistance to vertical contact pressure for various materials tested in this investigation to a design bearing capacity at 140 F and 0.05-in./min rate of loading for purposes of comparison. This also shows the feasibility and practicability of testing at 0.05-in./min rate of loading and at a temperature of 80 F and reducing the computed pavement resistance by a factor of 4 to determine bearing values for design. Table 19 gives the bearing capacity of various mixtures as tested in this investigation, and as adjusted if necessary to 140 F and 0.05 in./min.

The values of bearing capacity as shown in Table 19 may be considered ultimate bearing capacity and have not

TABLE 18

STRENGTH OF ASPHALT CONCRETE CONTAINING TWO PERCENT ASBESTOS FIBERS[a]

Property	Strength
Marshall stability, lb[b]	1,895
Marshall flow, 0.01 in.	17
Flexure (MR), psi	122
Unconfined compression, psi	158
Triaxial compression, psi:	
15-psi lateral	208
30-psi lateral	257
ϕ, deg	32.5
c, psi	44
Pavement resistance, psi	690

[a]Average of triplicate specimens for gradation B, 6.0 percent asphalt (85-100 penetration), 0.05-in./min rate of loading, 80 F.
[b]Marshall test at 140 F and 2.0 in./min.

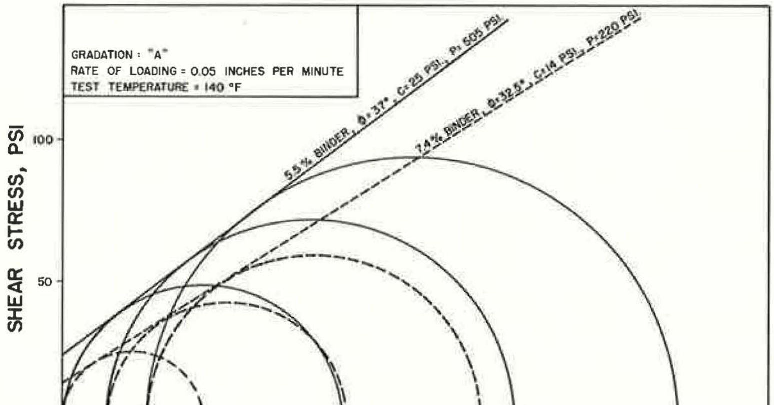

Figure 26. Mohr diagram for Viadon at two binder contents.

TABLE 19

DESIGN BEARING CAPACITY FOR VARIOUS MIXTURES

Aggregate Gradation	Binder		Test		Bearing Capacity		
	Type	%	Temp (° F)	Load Rate (in./min)	Test (psi)	Test Factor	Design[a] (psi)
A	A-C 629, A.C., flake	5.5	140	0.05	660	1	660
A	Viadon	5.5	140	0.05	505	1	505
A	Viadon	7.4	140	0.05	220	1	220
A	15% flake	5.5	140	0.05	200	1	200
A	25% flake	5.5	140	0.05	255	1	255
A	85-100 pen.	6.0	140	0.05	145	1	145
A	85-100 pen.	6.0	140	0.5	310	2	155
A	85-100 pen.	6.0	80	0.5	560	4	140
A	85-100 pen.	6.0	110	0.5	300	2	150
A	154 pen.	5.5	80	0.05	580	4	145
A	58 pen.	5.5	80	0.05	720	4	180
A	12 pen.	5.5	80	0.05	2,270	4	568
B	85-100 pen.	5.75	140	0.05	157	1	157
B	82 pen.	4.0	80	0.05	508	4	125
B	82 pen.	5.0	80	0.05	565	4	140
B	82 pen.	6.0	80	0.05	553	4	138
B	82 pen.	7.0	80	0.05	482	4	120
B	82 pen.	8.0	80	0.05	418	4	105
B	2% asbestos 85-100 pen.	6.0	80	0.05	690	4	173

[a]For 140 F and 0.05-in./min unadjusted for traffic factor.

TABLE 20

PERFORMANCE RATING OF PAVING MIXTURES FOR HEAVY-DUTY HIGHWAYS

Traffic Factor, T	Bearing Capacity (psi)	Performance Rating
1.0	75	Poor
1.5	115	Fair
2.0	150	Good
2.5	190	Excellent

TABLE 21

FLEXURAL STRENGTH OF VARIOUS MIXTURES[a]

Binder Type	Binder (%)	Flexure (MR) (psi)	Deflection (in.)
85-100 penetration asphalt	6.0	14	0.050
15% flake, 85% asphalt cement	5.5	8	0.055
25% flake, 75% asphalt cement	5.5	14	0.067
18% A-C polyethylene 629, 2% polybutene, 15% flake asphalt, 65% asphalt cement	5.5	60	0.055
100% A-C polyethylene 629	4.5	122	0.049
20% Tenite, 80% asphalt cement	6.0	72	0.028
20% Tenite, 80% flake asphalt	10.0	66	0.019
40% Tenite, 60% flake asphalt	8.5	165	0.025
Portland cement	-	955	0.024

[a]Gradation A, 0.05-in./min rate of loading, 140 F.

been subjected to a safety factor or traffic factor. To determine the suitability of these various mixtures to roadway requirements, it would be necessary to consider design loads and pavement category. For instance, if a pavement is for a heavy-duty highway where contact pressures are 75 psi, it would be essential that the design bearing capacity be 150 psi, as computed in Table 19, if the pavement is to have a traffic factor of 2.0. Similarly, it might be expected that a paving mixture having a bearing capacity of 75 psi would show early surface deformations if placed on a heavy-duty highway. A means of rating the expected performance of paving mixtures for heavy-duty highways on the basis of bearing capacity is given in Table 20.

There are problems in the design of flexible paving mixtures other than those relating to shear strength. Of considerable importance are the flexibility of the pavement layer (its ability to deform in flexure without cracking) and the ability of the pavement to resist tensile stresses with changes in environmental conditions without cracking. A flexible paving mixture cannot be designed to resist flexural stresses without reinforcement if excessive flexural deflections are permitted. The problems related to the flexural and tensile stresses and deformations of flexible paving mixtures should be the subject of an intensive investigation. Most of the research relating to pavement strength as applied to design practice has been in the area of shear strength. Research on the flexural or tensile strength of paving mixtures has not been extended to a consideration of design and it is in this area that work needs to be done (11, 12, 13, 14).

Table 21 gives a comparison of the flexural strength and deflection at failure for various mixtures. This is probably of more interest for a comparison of the flexibility of these mixtures as measured by deflection than by modulus of rupture.

CONCLUSION

Instrumentation has been developed for specimen preparation and the testing of flexible paving mixtures in flexure, tension, and triaxial shear. The influence of test temperature and rate of loading on asphalt concrete tested by these procedures was shown. Various mixtures were tested and analyzed for suitability to resist compressive stresses in the pavement. The practical application of this procedure to design is shown.

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Discussion

W. H. CAMPEN, Manager, Omaha Testing Laboratories —Asphalt paving mixtures must resist displacement to prevent shoving and rutting when subjected to the application of loaded tires. It may be possible to measure this resistance by shear but it should be pointed out emphatically that stability tests such as the Marshall and the Hveem do measure this resistance fairly accurately. For instance, it is well known that a Marshall stability of about 2,000 lb does prevent shoving and rutting under heavy truck traffic.

I might add that in my part of the country, shoving and rutting are a thing of the past. The change has been brought about by using stability tests as the principal criteria for design. Furthermore, stability has been correlated with type of traffic for economic reasons. Marshall stabilities of about 1,000 lb for light traffic, 1,500 lb for medium traffic, and 2,000 lb for heavy traffic are sufficient.

WILLIAM L. HEWITT, Closure—Flexible paving mixtures provide resistance to plastic deformation through intergranular friction and through the internal resistance of the binder to shear stresses. Shearing and rutting are forms of plastic failure in the pavement. The pavement component fails in compression while laterally supported. The triaxial test provides a good measure of the resistance offered by paving mixtures under these conditions. Marshall and Hveem stability values give some measure of the combined effect of intergranular friction and internal resistance of the binder, though Marshall stability may be influenced primarily by the internal resistance of the binder and Hveem stability may be influenced primarily by intergranular friction. Therefore, they too provide an indication of the resistance offered by paving mixtures to displacement when subjected to the application of loaded tires. The author is of the opinion that the triaxial test as described in this paper provides a better measure of resistance to the plastic deformation of paving mixtures than either Marshall stability or Hveem stability.

Pavement resistance values computed from triaxial data may be adjusted by a traffic factor to provide design values for various traffic conditions (Tables 19 and 20). This would serve the same purpose as varying Marshall stability for traffic category as suggested by M. Campen.