AN APPARATUS FOR LABORATORY INVESTIGATIONS

OF ASPHALTIC CONCRETE

UNDER REPEATED FLEXURAL DEFORMATIONS

by

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ABSTRACT

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AN APPARATUS FOR LABORATORY INVESTIGATIONS OF ASPHALTIC CONCRETE UNDER REPEATED FLEXURAL DEFORMATIONS

A laboratory study was performed for the evaluation of an apparatus constructed to investigate the behavior of asphaltic concrete specimens subjected to repeated flexural deformations.

The apparatus, called a deflectometer, was constructed to test flat circular specimens approximately 17 1/2 inches in diameter. The specimen under test was fixed about its periphery, a uniformly distributed pressure acted on the bottom surface to give the specimen a measured amount of support, and a repeated load which varied sinusoidally in magnitude with respect to time was applied to a centrally located area on the top surface of the specimen. The following is a list of loading variables that can be achieved with the apparatus:

1. Frequency of loading
2. Maximum amplitude of load applied
3. Ratio of maximum to minimum load applied
4. Ratio of load area to specimen area
5. Load contact pressure
6. Specimen support pressure

A standard loading condition was established for the purpose of studying various specimen characteristics with respect to their contributions to the ability of specimens to resist the effects of

repetitive loads. The effects of specimen characteristics were compared on the basis of number of load applications to cause failure of a specimen. The study indicated that the endurance of asphaltic concrete to repeated loads can be correlated with the following items:

1. Asphalt content of mixture
2. Asphalt consistency
3. Surface texture of aggregate
4. Stress or strain imposed on the specimen in consideration of
   (a) specimen thickness
   (b) support given the specimen
   (c) load contact pressure

Through the course of the study a mixture compaction method was developed to produce adequate density and particle orientation in the large test specimens. The compaction method utilized the testing machine to impart vibratory-kneading forces to the mixture being compacted. The compaction forces contained horizontal components which are believed necessary in order to tumble the particles and aid in obtaining proper densification of the specimen.

INTRODUCTION

The research reported here is concerned with factors that affect the flexibility of asphaltic surfacing mixtures, with the construction of a testing apparatus to evaluate these factors, and with a laboratory compaction method developed for producing large test specimens.

Within the last decade investigators of asphaltic surfacing and bituminous mixtures have become increasingly aware of the necessity for considering the flexibility and the so-called fatigue characteristics of asphaltic surfacing. The effects of these characteristics have been manifest in the surfaces of pavements built over resilient foundations or in roads subjected to high traffic volumes and weights.

Bituminous road surfaces have been labeled "flexible" surfaces, not because of lack of rigidity, but because of their ability to conform to slow changes of contours which occur within the foundation. The shifts of elevations within the foundation of a road may be brought about by volumetric changes of this supporting medium due to moisture content variations or by deformations caused by the passages of wheel-loads. The deformations of direct consideration in this thesis are those which are caused by the passage of traffic and which are immediately recovered; these are the more frequent and may cause failure of a bituminous surfacing as evidenced by cracking. Of course, other types of deformations lead to and constitute failure of a surface, for example, rutting, shoving, and raveling or eroding; but these are not within the consideration of this report.

Undoubtedly, shrinkage stresses set up by decrease of temperature must be considered in the analysis of pavement surface cracking. However, field observations indicate that cracking of asphaltic surfaces does not generally occur during the winter, but is most prevalent throughout the spring thaw when the foundation is water-saturated and in its weakest state. Also, it is known (13)[1] that the tensile stresses caused by shrinkage are at most one-half the value of the tensile strength of the asphaltic mixture.

Hveem (1) has indicated that the major portion of the deflection caused by a wheel-load occurs in the foundation soil; nevertheless, the repetition of a great number of small surface deflections can and does cause surface distress, generally termed "alligatoring" (Figure 1). A relatively small number of large deflections can be responsible for the initiation of this type of failure. Nijboer and van der Poel (13) make the following comment about these cracks:

> .... these cracks appear in a later stage of life of the carpet and a difference of opinion seems to exist on the importance that must be attached to its practical implications.
>
> Sometimes it is said that they seriously shorten the life of the carpet; sometimes they are considered as just beauty failures.

The distress of alligatoring must be considered of great importance both with respect to the maintenance of structural integrity of the total pavement and to the aesthetics of the surfacing. For these reasons procedures for the design of a bituminous surfacing material which is resistant to, and/or is able to withstand, repeated flexing without cracking must be established. It is believed that such a mixture-design

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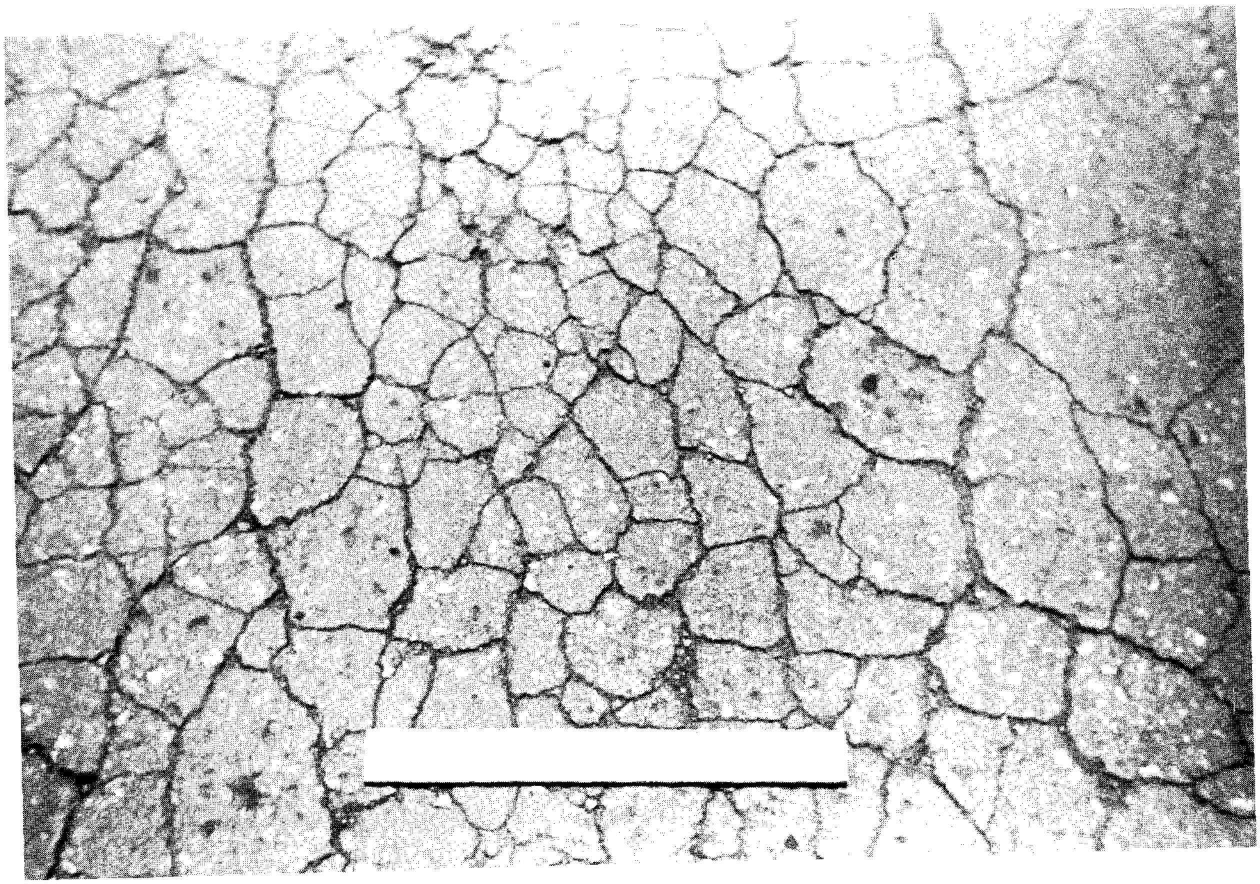
[1] The numbers in parentheses correspond to their listing in the Reference Citations.



A Pavement Surfacing Failure

Figure 1

procedure must originate from a semi empirical method due to the complex nature of the materials involved and of the stress conditions induced by loads on bituminous surfaces. Presently, all existing asphaltic surfacing mixture-design procedures are empirical, but the success of these designed mixtures has been dependent on prior service correlation. As the review of literature in this field shows, most of these mixture-design procedures do not employ repetitive load cycles in the evaluation of asphaltic surfacing materials.

In the past, the usual procedure used in testing for flexibility has been to load a prismoidal shaped specimen in some sort of beam condition. This type of testing does not appear to be adequate for duplicating service conditions, although it does serve to obtain comparative values of parameters with which to determine the effects of certain variables. A model for testing, more closely related to the conditions in the pavement, is a large diameter disc fixed about its periphery and loaded on its center; this is the model used in the research.

The forming of the large diameter (17 1/2 in.) specimens presented a problem in that no satisfactory method was available for this purpose. A vibratory-kneading method was devised that would compact the specimens to adequate density and with favorable particle orientation.

The remainder of this dissertation will deal with the development of a suitable apparatus for testing the flexibility of asphaltic surfacing mixtures under conditions similar to those existing in the actual pavement. The use of the apparatus for evaluation of asphaltic surfacing mixtures will be explored in considerable detail, and data obtained from the testing program will be presented.

REVIEW OF THE LITERATURE

Prior to 1950 not much had been written regarding the flexibility characteristics of asphaltic surfacing mixtures. During this period most investigations of the design of such mixtures were concerned primarily with stability measurements and requirements. Since then the importance of flexibility has been recognized and studied principally in Europe and in the United States. In the United States the need for emphasis on the flexibility of pavements was presented to the highway materials engineer by Hveem (1) in 1955.

This review spans the literature dealing primarily with the behavior of asphaltic concrete surfacing when subjected to bending or tensile stresses and with the mechanics of compacting and of testing the prepared specimens. It is recognized that the preparation or method of molding specimens and the testing procedure are important factors in determining and establishing the physical properties of asphaltic mixtures.

Rader (2,3,4) was one of the first to investigate the factors affecting the bending properties of asphaltic concrete. His investigations were concerned with determining the physical properties of asphaltic mixtures at low temperatures. Rader considered cracking of asphaltic pavements to be caused by shrinkage stresses brought about by reduction of temperature. The apparent interest was in obtaining values of tensile strength and modulus of elasticity of fine grained mixtures at temperatures below $0^{o}F$. These values were determined by testing prismatic specimens 2 inches wide, 1 1/2 inches deep, and 8 inches long under a simple beam condition. The specimens fabricated in the laboratory were compacted by tamping with the Hubbard-Field equipment and the application of a 3000 pound per square inch leveling load. The modulus of rupture (tensile strength) and a secant modulus of elasticity were computed by the use of formulae employed for elastic simple beams. From these early studies, Rader concluded that (a) the modulus of elasticity increased as the density of the specimens increased (within limits), (b) a mixture containing a high penetration grade asphalt should be more resistant to cracking at low temperatures, and (c) a desirable mixture should have a low modulus of elasticity and a high modulus of rupture to resist thermal stresses. Today, however, there appears to be general agreements that temperature induced stresses are not the cause of pavement surface cracks since cracking occurs primarily during the springtime rather than during winter. Rader's investigations are important in that they pointed out the need for testing asphaltic concrete under flexural stresses.

Following Rader's lead and using his basic methods, Rashig and Doyle (5,6) tested prismatic specimens under cantilever beam conditions. The purpose of their study was to compare moduli of rupture and elasticity of sheet asphalt mixtures made from fourteen different asphalts. One of the conclusions reached was that, for the same penetration grade of asphalt, there was not a great variation in the beam tests for the asphalts studied.

Hubbard and Field (7) proposed a method based on deformation by which the thickness of an asphaltic surface could be determined. This thickness was established by loading a circular area on a 22 inch square of asphaltic concrete supported by a compacted soil. The strength of the asphaltic concrete was determined at the load causing 0.5 inch deflection of the circular loading head. The specimens were compressed by hand tampers.

Advancements in the methods of testing for flexibility through the use of beams were made by Hillman (8) in 1940, who was at that time with the Bureau of Public Roads. One of the variables included in Hillman's study was the rate of load application. Data were presented to show that increasing the rate of loading increased the measured strength and stiffness of the beam specimens. As previous investigators had done, Hillman computed the moduli of rupture and elasticity under the assumption that the specimens behaved elastically when subjected to load. Of particular interest were the load-deflection values showing that although the central forces causing failure of identical beams varied with the rate of loading, the center deflections of the beams were practically equal at failure. It is reasonable to assume that this center deflection was the critical factor in determining the load carrying capacity of the beams.

In 1948 Thomas (9) reported his studies on testing 16 inch diameter asphaltic concrete specimens. These specimens were supported on a spring base and loaded centrally over a concentric surface 4 inches in diameter. Eight repetitions of a unit pressure of 200 pounds per square inch were applied, and then the specimen was turned over and eight more repetitions of the load were repeated. The strength of the specimen was determined by its ability to withstand the loading cycle and the amount of damage caused by testing. Thomas concluded that the stiffer mixtures, in spite of their greater load distribution characteristics, may not have a real advantage since they may be more susceptible to cracking.

At the University of Washington, Chen and Hennes (10) investigated the flexural resistance of asphaltic concrete mixtures. A prismatic specimen completely supported on a steel leaf spring was loaded by a rotating eccentric disc in contact with the specimen at a rate of 37 times per minute. Different magnitudes of loads to cause failure were obtained by varying the eccentricity of the rotating disc. A plot of dynamic modulus of rupture (arithmetic scale) versus repetitions of load to produce failure (log scale) showed a linear relationship. The extrapolation of this curve was used to obtain the strength of the mix at one repetition of load, and this value was used to determine dimensional requirements for the structural design of a pavement. In this report a dynamic modulus of elasticity of 30,000 pounds per square inch was stated for the mixture as determined by vibrosonic methods.

Hughes and Faris (11) investigated the deformability of asphalts and of mixtures of asphalts with sand, under high and low deformation rates. The high deformation rate of 3 inches per second did not differentiate among the various asphalts tested as asphalt-aggregate mixtures; specifically the effects of this high rate of deformation masked the effects of crude source, penetration value, and temperature on the flexibility of the specimens. The low rate of deformation of 0.07 inch per hour yielded results in the anticipated directions when the contributions of penetration and temperature to the asphalt's resistance to load were considered.

Van der Poel (26) in 1951 described a vibration machine for applying dynamic loads of sinusoidal variations with time to actual pavements. This machine was used to determine the propagation of waves through the pavement system and for evaluating the pavement stiffness. Field measurements of strains indicated that asphaltic surfaces may be stressed to values from 10 - 20 kilograms per square centimeter (140-280 psi) in tension. This machine has been utilized in the United States by such agencies as the California Division of Highways, the Bureau of Public Roads, the Highway Research Board, and the Corps of Engineers. Izatt, Evans, and Metcalf (12) have summarized the results of the testing done in the United States with the machine.

A study by Rigden and Lee (14) on the brittle fracture of tars and bitumens showed that the apparent tensile strengths of these materials reached constant values as the rate of loading was increased and as the test temperature was lowered. Although the tensile strength of a bituminous mixture is largely dependent on the tensile strength of the binder, it is not apparent that one rate of loading and one test temperature may reasonably be established for testing these mixtures under dynamic loading conditions in the laboratory.

The flexural strength of asphaltic concrete specimens was determined in the WASHO Road Test (15). Sonic moduli of elasticity were found over a span of temperature from 0 to 80°F, with values ranging from 1,500 to 3,500,000 pounds per square inch. Moduli of rupture were determined by the employment of a high rate of loading (the period of load approximated 1/5 second). A plot of modulus of rupture versus test temperature showed an increase in strength with increase in temperature up to about 60°F and then a decrease at higher temperatures.

Monismith et al (16,17,18) have presented several papers on the flexibility of asphaltic surfacing mixtures. The test specimens were tested on a flexible spring base support similar to that used by Thomas (9). Repetitive loads were applied to the specimen by means of a pneumatically operated device developed at the University of California at Berkeley. The specimens were subjected to the repetitive loads on the device mentioned, but these loads were not intended to rupture the beam specimens. After various numbers of repetitions of loads, the specimens were tested in simple beam loading for the determination of modulus of rupture. A number of variables, including load and asphalt content, were compared on the basis of retained strength (modulus of rupture) after certain numbers of flexing load applications. In the first report (16) Monismith showed a fairly complete enumeration of the factors to be evaluated in order for one to understand better the property of flexibility of asphaltic mixtures. The major factors may be summarized as (a) properties of asphalt, aggregate, and mixtures, such as, viscosity, gradation, and density, (b) structural effects, for example, slab thickness and load distribution, and (c) external condition, that is, temperature, support, and loading factors. Monismith et al have concluded that the fatigue behavior of the mixtures tested was dependent on (1) the amount of asphalt, (2) magnitude of the repeated load, (3) the gradation of the aggregate, and (4) the test temperature.

Goetz, McLaughlin and Wood (19,20) have reported on studies made at Purdue University on load-deformation characteristics of asphaltic concrete. The testing was primarily on cylindrical specimens subjected to axial loads. Variables investigated were temperature, rate of loading and confining pressure for triaxial compression specimens. Data presented indicated a logarithmic relationship between permanent deformation and number of load repetitions until failure of the specimen was attained. A general equation was derived which relates unconfined compressive strength of a mixture to temperature and rate of loading. The equation was as follows:

$$X_o = A^{BX_1^{(CX_2 + D)}}$$

where:

X_o = maximum unconfined compressive stress, psi

X_1 = rate of deformation, in/min.

X_2 = temperature, ^oF

A,B,C,D = constants of proportionality.

It has been suggested that the asphalt viscosity be substituted for temperature in the above equation.

The fatigue properties of asphaltic concrete were investigated by Papazian and Baker (21) utilizing repetitive loads on prismatic specimens. The specimens were tested as simple beams with an elastic support at the center. It was concluded that:

> the number of repetitions of load which a pavement may undergo without cracking appears to be a function of the stress level, the function being of a linear type on a double-log scale.

Ekse (22), in studying the influence of amount of filler upon the flexural strength of asphaltic concrete, tested slabs 11 inches square. A slab was simply supported about its periphery and loaded centrally on a 2 inch diameter bearing area. Data were presented to show the great influence that specimen thickness has on flexural strength and also that maximum strength was fairly consistently obtained when the center deflection reached approximately 0.13 inch.

Saal and Pell (23) obtained the effects of repeated stresses on a sheet asphalt mixture. Specimens were tested as cantilever beams for constant amplitude of stress and as simple beams for the condition of constant bending strain. Other variables studied were speed of load application, temperature, and amount of voids in the specimens. The conclusions reached were as follows.

1. Fatigue life depends on temperature and speed of loading.
2. An increase of void content decreases the fatigue life.
3. The superposition of a constant tensile stress on an alternating bending stress decreases the fatigue life, and a superimposed compressive stress increases it.
4. There was no evidence of an endurance limit.
5. The dominant factor affecting the fatigue life was strain.

The preceding review has been concerned primarily with the behavior of asphaltic concrete mixtures when subjected to flexural stresses. Of primary importance to this writer have been the methods used to compact test specimens and the shapes and support given the specimens during testing operations. These factors were studied critically because it was believed that the specimen under test conditions should be restrained and perform in a manner similar to actual road conditions. None of the cited methods fulfilled this requirement or condition. One further requirement should be that the test conditions have analytical or so-called "rational" expressions for stresses, as opposed to purely empirical equations, to define specimen characteristics. The development of a machine and method of test which approaches the fulfillment of these conditions is described in the following section.

DEVELOPMENT OF THE NEW TESTING APPARATUS (DEFLECTOMETER)

In the design of any structure the engineer must have some preconceived values of the physical properties of the materials proposed for the structure and of the anticipated forces that must be resisted, be these imposed by Man or Nature. A measure of the adequacy of a material to resist these forces sucessfully is ordinarily obtained in the laboratory. Laboratory measurements, by themselves, are not conclusive but when tempered with experience have proven to be reliable guides.

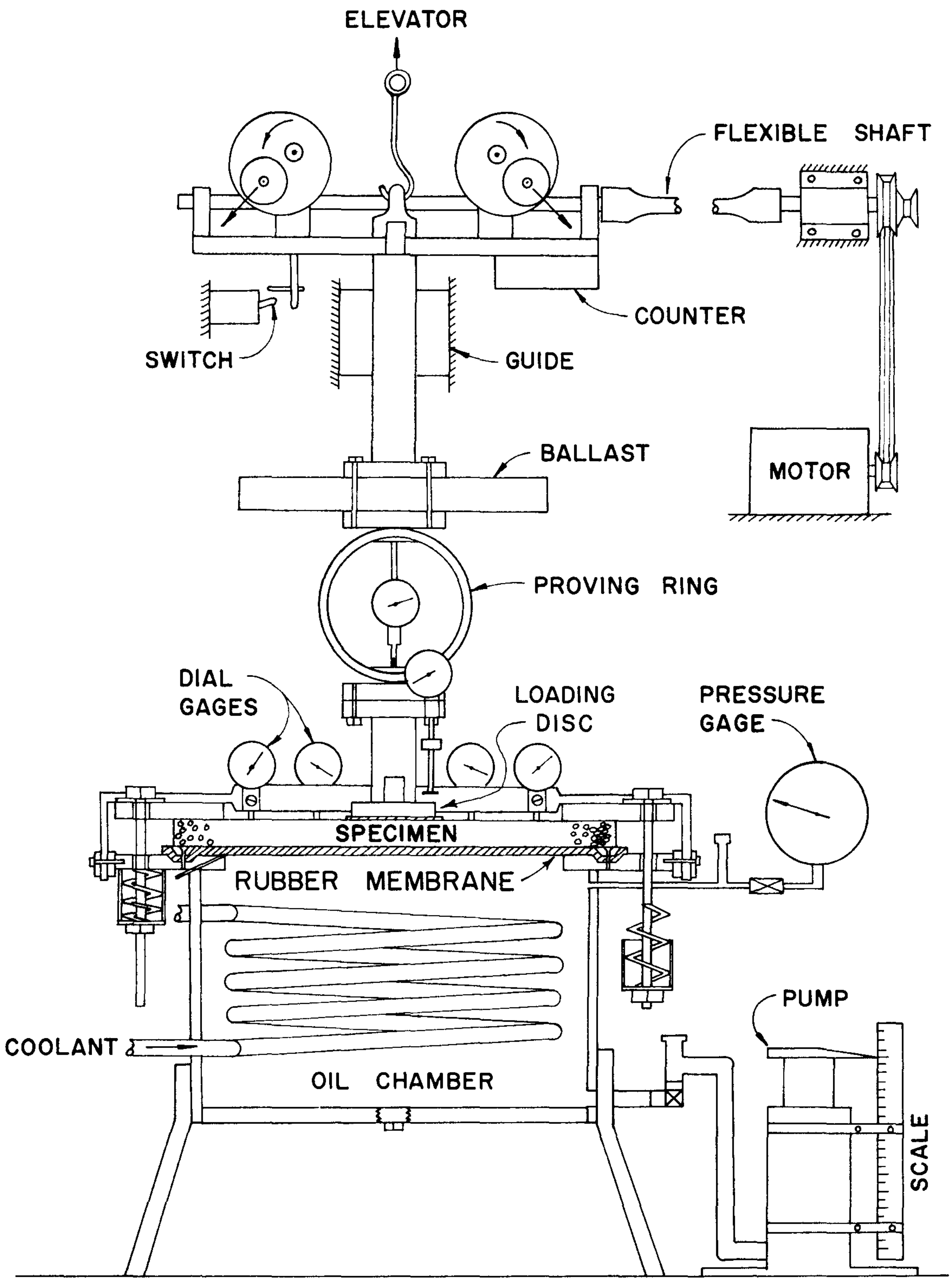
In the design of bituminous surfaces it is virtually impossible to duplicate completely the actual service atmosphere in the laboratory; for a fact, it would not be desirable from the standpoint of economy of testing. Nonetheless, the challenge of actual service testing has been met by such test roads as the Bates, Stockton, Maryland, WASHO, and AASHO in the United States. The desire to duplicate actual road conditions and yet control certain actions of Nature has led to the construction of miniature road sections - minitracks - by interested agencies (24). In these tests several variables are being investigated simultaneously. For the everyday, routine type of test, one variable at a time is usually investigated, and for this purpose the test method must be relatively simple and unsophisticated.

In order to learn about the bending properties of asphaltic road surfacings, it was necessary to build an apparatus whose primary purpose would be to evaluate in the laboratory the flexibility and resistance to cracking of an asphaltic concrete slab. It was not intended that this apparatus be used for bituminous mix design, although the possibility for this usage was considered. It has been known and aptly stated by Please (27) that:

> No one test can predict the ability of a material to satisfy the two requirements of continued resistance to deformation and continued resistance to fracture or disintegration so that if a complete assessment is required of the performance of a bituminous surfacing material, separate tests must be made to determine:
> (a) its resistance to deformation;
> (b) its flexibility and resistance to fracture; and
> (c) the effect of weather on these properties.

A schematic diagram of the device designed purposely for testing asphaltic concrete in the shape of large circular specimens is shown in Figure 2. It can readily be seen that the machine has two primary components, a loading system and a reaction assembly. These two components are described as follows.

In the loading system it was thought to be advantageous to have a constant force (dead load) which could be varied by the addition of ballast, and a fluctuating force (live load) which would add to and subtract from the dead load. The live load was obtained by the rotation in opposite directions of two eccentrics. This system is similar to an oscillator (25) used for determining the frequency of free vibrations of structures; also the road vibrating machine described by van der Poel (26) utilizes a similar vibrator except that three eccentrics are used. Since the masses of the eccentrics are balanced and both have identical rotational speeds and "mirror" positions, their net force is one of vertical direction (the horizontal components are equal in magnitude and of opposite sense) and varies in a sinusoidal fashion. The force exerted by the loading system may be varied in amplitude by adding to the



SCHEMATIC DIAGRAM OF DEFLECTOMETER

NO SCALE

Figure 2

mass of the eccentrics or by increasing the speed of rotation. In order to prevent impact (i.e. collision) the maximum live load must be less than the dead load. The total maximum live load is the sum of the forces induced by the rotation of the eccentrics plus the inertia force caused by the translation (acceleration) of the dead load in deflecting the specimen and proving ring.

A specifically desired feature of the loading system was that the same magnitude of load should be maintained on the specimen throughout the period of testing. That is, as the specimen underwent plastic deformation, the loading system would follow the movement. To accomplish this end, the eccentrics were driven with a flexible shaft.

Investigations of variations in contact pressure were made possible by interchanging loading discs of different diameters.

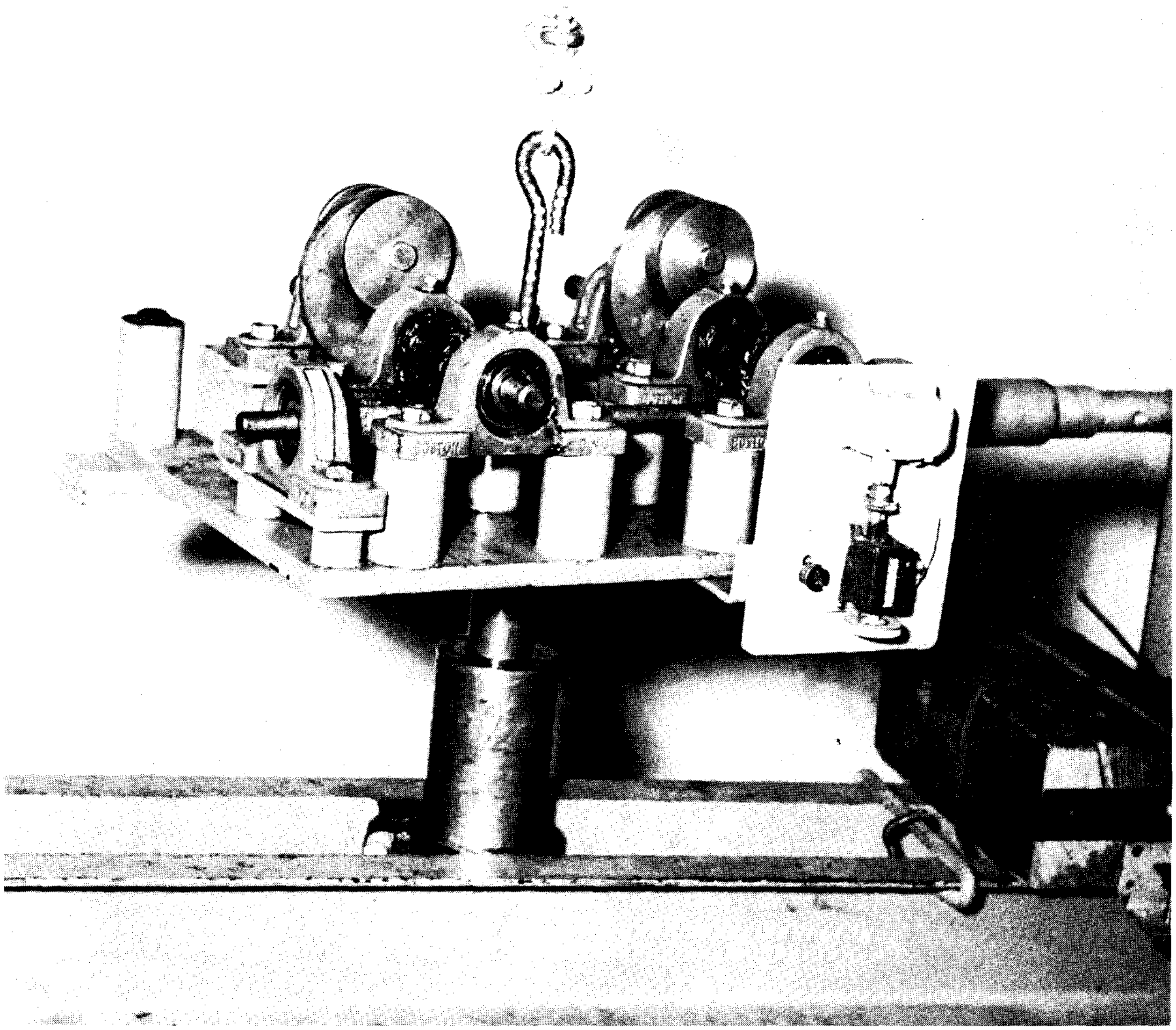
The reaction unit consisted of a cylindrical oil chamber sealed at the top with a thin rubber membrane and to which was connected an oil pump. A specimen was clamped onto the oil chamber by means of a steel ring and bolts. The clamping force of each bolt was controlled by the use of a spring and a metal spacer sleeve. The bolt was drawn until the spring was compressed and the spacer just made contact with the bottom surface of the shoulder on the chamber. All of the bolts had identical springs and sleeves of the same length.

With a specimen in place, the oil pressure in the reaction unit could be varied by displacement of the pump piston. This afforded a means of controlling the initial support given to a test specimen. A bourdon gage indicated the uniform pressure acting on the bottom surface of the specimen. With the arrangement described, it was possible to control or adjust the amount of air in the system. This control of air was necessary since a pressure build-up would be created as the specimens deformed under loading, and it was required that the compressibility or yielding characteristics of the support be constant and measurable. The adjustment of air in the system (calibration) was achieved by clamping a stiff metal disc onto the chamber and correlating the linear displacement of the pump piston with the increase of pressure in the oil chamber. During testing operations the pump was disconnected from the reaction unit by means of a valve.

Accessories to the reaction unit were a dial gage carriage and internal coils for the heating or cooling of the oil in the chamber. The dial gage carriage that was secured to the shoulder of the oil chamber gave a fixed reference for the measurement of deflections on the upper surface of the specimen.

It does not appear to be necessary to cite specifically the dimensions and weights of the components of the loading system since similar combinations will produce comparable ranges of loading capacity. The particular loading apparatus used for this investigation is illustrated in Figure 3. It can be noticed that the directional change of rotation from the flexible shaft to the eccentrics was achieved by means of 45° helical gears, one pair being right-handed and the other left-handed. Dimensional limitations precluded the use of spur gears. With this design, it was not possible to rotate the eccentrics smoothly at angular speeds much below 500 revolutions per minute.

As with the loading system, the dimensions of the reaction unit are not required to be of fixed lengths; the importance is in the



Loading System

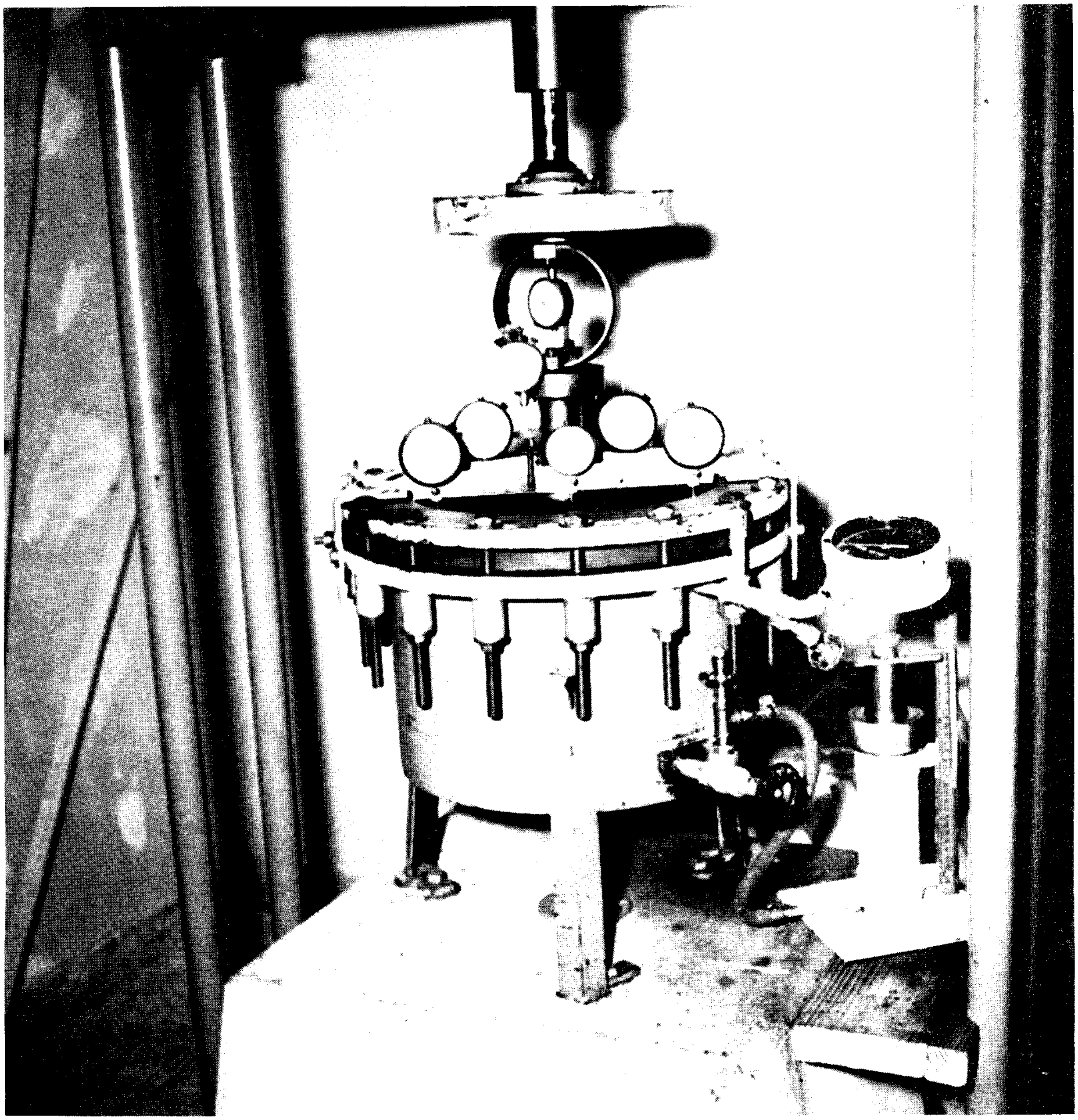
Figure 3

support given to the specimen and the calibration of the air-oil system. Calibration procedures used are explained in Appendix A. A photograph of the reaction unit is shown in Figure 4.

Dimensions of more interest are those of the specimens and loading discs, which established ratios of loaded area to specimen area. Specimens were molded in a short length (4 in.) of 18 inch pipe. This operation resulted in a specimen approximately 17 1/2 inches in diameter, although not exactly round. The divergence from a circular shape was immaterial since it was not intended for the specimens to fit into a cylinder, in fact polygonal shaped specimens, such as trimmed road samples, can be tested. During a test the specimen was clamped onto the reaction unit about its periphery so that the net or effective diameter of the sample was 14 inches.

Three metal discs were available to load the specimens. The diameters of these were slightly less than diameters corresponding to areas of 3.14, 5.00 and 8.00 square inches. Secured to plane surfaces of the loading discs were circular pads of 1/8 inch thick rubber whose areas were those listed above. This arrangement was necessary in order to minimize the high perimeter shear associated with rigid bearing plates. The ratios of diameter of load disc to effective diameter of the specimen are thus 0.142, 0.180, and 0.228 for the 3.14, 5.00, and 8.00 square inch discs, respectively.

Studies using the Benkleman beam for the measurement of flexible pavement deflections by Dunlap and Stark (28) and a report by Hveem (1) show that the surface length of the area depressed by a 9,000 pound wheel load may vary considerably, from values as low as 8 feet to as



Reaction Unit

Figure 4

much as 18 feet. An average value for the contact length of a 10.00 x 20 tire rated at 5000 pounds and inflated to 80 pounds per square inch is 12 inches (29). Since a wheel load of 9000 pounds would necessarily be carried by dual wheel assemblies, it has often been assumed that the equivalent area of tire contact is a 12 inch diameter circle. Under these conditions the ratios of load diameter to deflected surface length occurring in the field vary from 0.125 to .055. The disparities between the ratios of loaded areas to deflected surface areas for the apparatus and actual field loading are not as great as they seem to appear since it has been shown (15) that the transverse length of a deflected pavement surface is not as great as that in the longitudinal direction. Considering the transverse direction, these ratios could be from 0.10 to 0.25.

The effects of the ratio of loaded area to the size of specimen on the load-carrying capacity of asphaltic concrete slabs were studied by McLaughlin (30). This study showed that load-carrying capacity of rigidly supported specimens increased as the ratio of specimen area to loaded area increased until this ratio was approximately 10, and then the strength of the slabs remained essentially constant. It can be seen that this ratio has been exceeded for all three loading discs since the ratios were 19, 31 and 49 for the 8.00, 5.00 and 3.14 square inch discs.

In a previous section, mention was made of the fact that it was desirable for the test conditions to yield results capable of analytical expressions for the stresses imposed on the specimen. The specimen as tested is a circular plate fixed about its circumference with a uniform pressure acting on the membrane at the bottom surface and with a central load on its upper face. According to Morley (31), Grashof derived equations for stress and strain in a circular plate by use of the Bernoulli-Euler theory of bending. The solution assumes that the material behaves elastically and, therefore, it is subject to certain limitations where localized yielding or flow occurs, which result in a redistribution of stresses. Further, the theory of bending employed was based on a homogeneous and isotropic material. Generally, the stresses obtained from such flexual formulae are based on static loadings and do not include the inertial forces created in moving the material, as occurs in dynamic testing. Allowances for deviations from the theory were made on the following basis: (a) homogeniety and isotropy were assumed to exist in a specimen as a whole since segregation was eliminated as much as possible, (b) elastic behavior probably existed for the high rate of loading employed (21), and (c) inertial forces within a specimen were of minor importance. The equations used for the determination of stresses and moduli of elasticity for a circular disc fixed about its periphery are shown below:

radial stress at the surface center,

$$S_x = \frac{3(m+1)W}{2\pi mt^2}\left(\ln\frac{r}{r_o} + \frac{r_o^2}{4r^2}\right) - \frac{3(m+1)r^2}{8\,mt^2}\,p \qquad (1)$$

and modulus of elasticity

$$E = \frac{3(m^2-1)}{4\,\delta\,m^2t^3}\left[\frac{Wr^2}{\pi} - \frac{(r^2-x^2)^2}{4}\,p\right] \qquad (2)$$

where m = reciprocal of Poisson's ratio

W = central load, pound

t = specimen thickness, inch

r = effective specimen radius, inch

r_o = radius of loaded area, inch

p = support pressure pounds per square inch

δ = deflection of specimen at x, inch

x = radial distance from center, inch

The value of Poisson's ratio for the mixtures was assumed to be 0.2 (m = 5). The central load, W, for the computation of radial stresses was the maximum load applied; for the determination of modulus of elasticity it was the difference between the maximum load applied; for the determination of modulus of elasticity it was the difference between the maximum and minimum central load. For the set-up used, r was 7 inches and r_o varied with the area of load disc. All calculations were made for the condition of p equal to 1.5 pound per square inch.

The equation for radial surface stress at the center was simplified by omitting the term $\frac{r_o^2}{4r^2}$ since this value was very small even for the largest loading disc. With the substitution of the constants used in testing with the 5.00 square inch load-disc, equation 1 simplified to,

$$S_x = \frac{201 + 0.97}{t^2} F_t \tag{3}$$

where F_t was the inertial force due to translation of the loading system.

The modulus of elasticity computed was in effect a secant modulus and will be called Dynamic Modulus of Elasticity since the deflection, δ, utilized for its determination was the repeated deflection observed and not the accumulated deflection at the time the support pressure reached 1.5 pound per square inch. It appears most probable that the value of this repeated deflection included a very slight amount of flow deformation. Reduction of equation 2 was obtained by allowing the $\underline{x}$ term to be zero and assuming that the repeated deflection of the loading disc was equal to the repeated deflection at the center of the specimen. The simplified equation for Dynamic Modulus of Elasticity computed from central deflections was then,

$$E_D = \frac{11.2}{t^3} (150 + 2F_t) \tag{4}$$

where $\underline{F_t}$ was the same as defined previously for equation 3.

It should be noted that the value of radial stress, equation 3, was dependent on the stiffness of the sample since specimens of equal thickness may not allow translation of the loading system to the same degree under conditions similar to the ones employed in this method.

The nomograph of Figure 5 was used to determine the amounts of the dynamic forces imposed on test specimens. The force due to the rotation of the eccentric was defined by the equation

$$F_R = Me\,\omega^2 \tag{5}$$

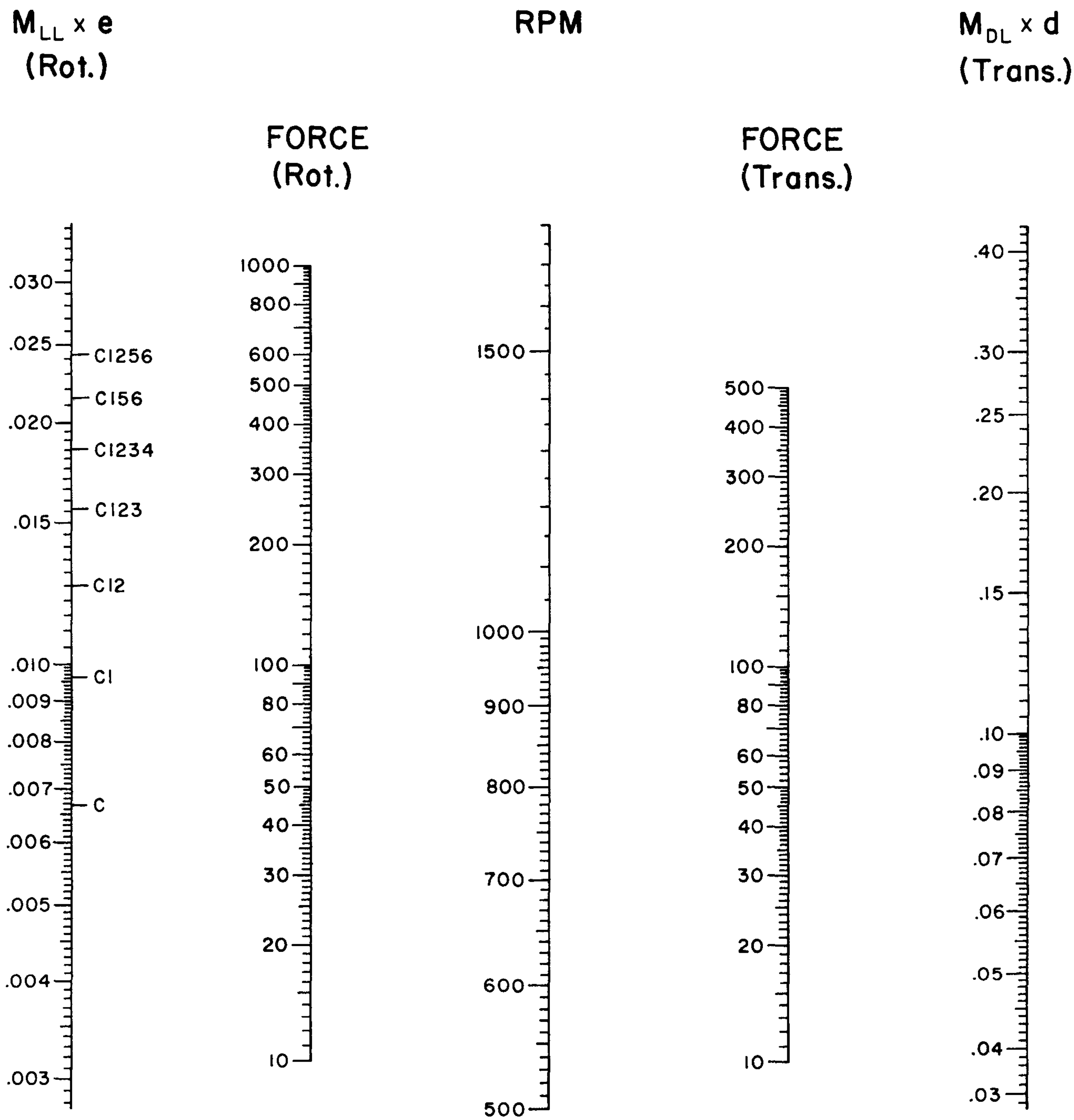
where F_R = centrifugal force, pound

M = mass of the eccentrics, slug

e = radius of rotation, foot, and

ω = rotational speed, radians per second.

In the nomograph the values on the right side of the M_{LL} x e axis (C,C1, etc) are products of mass times radius of rotation for the eccentrics used; $\underline{\omega}$ is represented in terms of revolutions per minute. The force created by revolving the eccentrics was found in the nomograph by the intersection of a line connecting the M_{LL} x e and the RPM axes with the $\text{Force}_{(Rot)}$ axis.



NOMOGRAPH FOR DETERMINING LOAD ON SPECIMEN

[LOAD = FORCE OF ROTATION (F_R) + FORCE OF TRANSLATION (F_T)]

Figure 5

The force imposed by the acceleration of the dead load was determined from the equivalent simple harmonic motion expression, which results in a formula similar to equation 5 and whose symbols are the same as described above. The force of translation is found in the nomograph in the same manner as for the force of rotation, but the term M_{DL} x d was obtained by multiplying the dead load mass times one-half the amplitude of movement of the dead load in inches. The maximum dynamic force is then the sum of the two forces found by use of the nomograph.

Before the testing program could be started, specimens of adequate density and structural characteristics had to be fabricated. This phase of the research is described next.

ESTABLISHMENT OF MOLDING PROCEDURE

The purpose of laboratory compaction of an asphaltic concrete mixture is to produce a specimen in such a stable state that it may be loaded to determine its strength. The mere fact that field densities of asphaltic concrete mixtures can be duplicated in the laboratory is not sufficient. Researchers (32,33) in the field of asphalt paving technology have known for a long time that the structural arrangement of the aggregates plays an important part in determining the strength of these mixtures. For the assessment of the physical properties of laboratory specimens it is imperative that the particle orientation of laboratory compacted samples be similar to that achieved in field compaction.

Various methods for compacting the large 17 1/2 inch diameter specimens were given consideration. The primary objections to manual methods were the physical exertion that would be required and the fact that different operators would produce individual variabilities. Presently, it is generally acknowledged that a "kneading" action is necessary in a laboratory compaction method for asphaltic concrete specimens. A kneading compactor imposes horizontal forces on a mixture which are required to obtain desirable particle orientation. It is not implied that rodding or tamping of loose asphaltic concrete mixtures does not give a kneading action to the mix.

The technical literature on the compaction of large specimens is extremely limited. The method developed for the molding of the flexure test specimen should be economical, mechanized, and capable of forming 4-inch diameter specimens for standard testing. The Dorry machine employed by Kriege (46) was considered, but it was anticipated that such a machine would be difficult to find or expensive to construct; further coring of the large diameter specimens would be necessary in order to obtain 4-inch samples. The same objections were raised in the consideration of the Road Research Laboratory's roller-compaction machine (27). The most acceptable method found in the literature review was that employed by McLaughlin (30) in compacting slabs of asphaltic concrete 16 inches square. A pneumatic vibrator with a 3 x 6 inch base plate was utilized by McLaughlin to obtain specimen densities up to 146 pounds per cubic foot.

Vanderlip (34,35) compacted 18-inch diameter specimens with a small motor-operated plate vibrator. The use of high asphalt content and well graded glacial material resulted in adequate densities with a minimum of effort and "... which in this respect (effort)[1] we were not able to duplicate with crushed material."

The mechanical vibration method for compaction of Calderon (36) did not appear directly adaptable to molding large asphaltic concrete specimens.

Indications from the review of the literature were that a vibrational compaction method would be required to obtain adequate densities. The obvious method of using the new testing apparatus for compaction was not overlooked. Objections to the use of the loading system to impart compactive effort for molding purposes came primarily

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[1] Parentheses are the writer's.

from the fact that the machine had been designed for testing and not for compacting. Various trials with a vibrating table failed to yield adequate densities in a fine grained but highly surface-textured mixture proposed for the testing program. High frequencies, up to 4200 revolutions per minute plus a low surcharge weight on the sample did not produce specimens of the required density. The use of direct compression (1200 psi) after rodding the mix also resulted in low densities. The inability of the above methods to product the necessary density led to experimentation with the loading unit of the proposed testing machine.

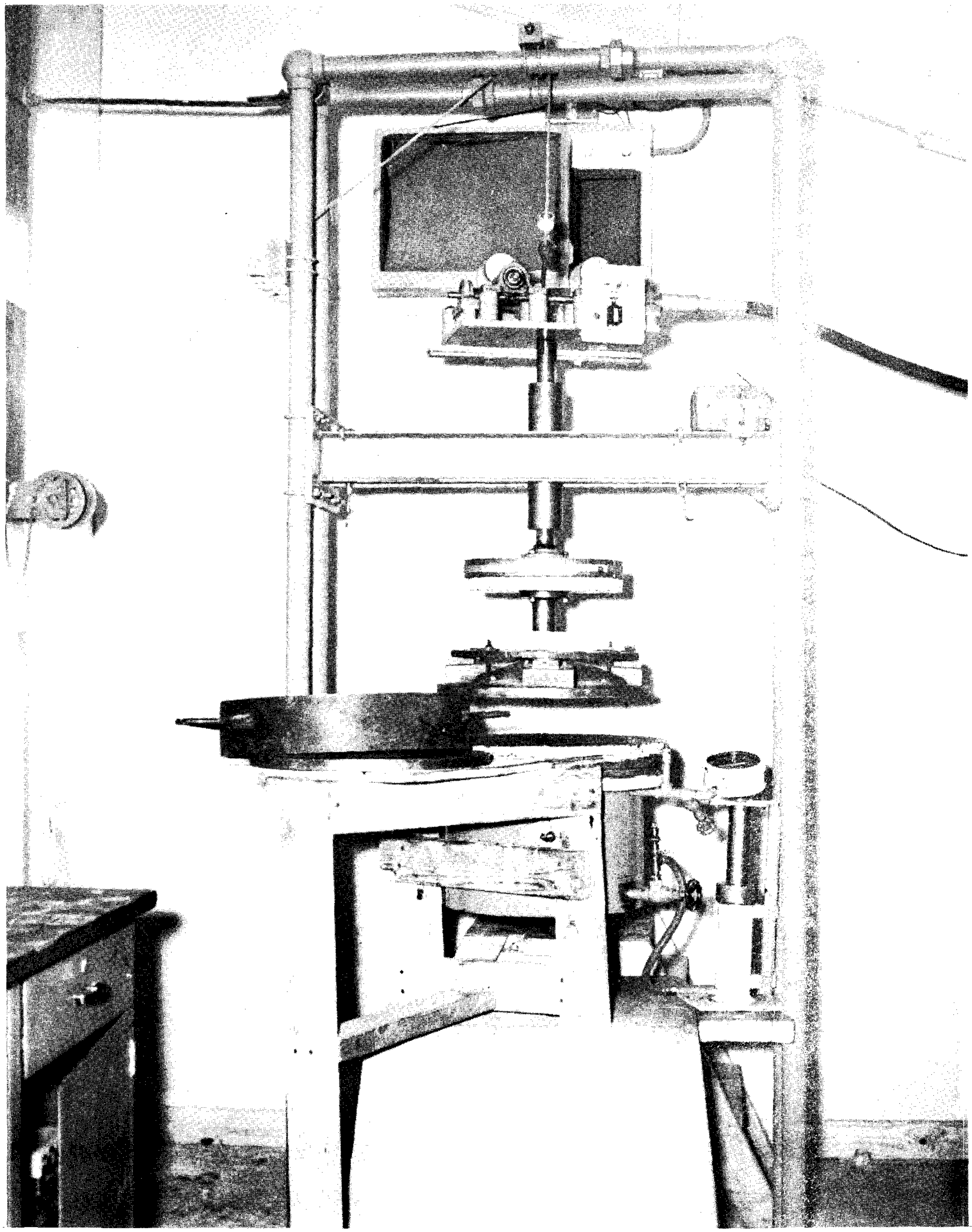
The criterion used for the comparison of densities of compacted asphaltic mixtures was that the densities of the large specimens should be at least 95 percent of the density obtained by compacting 4-inch diameter specimens by the Texas Gyratory-Shear method (37,38). This method imparts a kneading action to the mixture being molded, and compactive effort is applied until the densified mixture has a certain resistance to load. For a paving mixture of high internal friction the compactive effort required to reach the specified end point is greater than for one of low internal resistance to load. It has been shown (39) that this method of compaction is adapted to producing compacted specimens of variable heights but of equal densities for any one mixture of asphaltic concrete.

In the search for a compaction procedure utilizing the testing machine which would yield acceptable densities, the following factors were investigated: (a) frequency of vibration, (b) magnitude of dead load, (c) magnitude of live load, and (d) duration of compaction period. Actually, two procedures had to be developed, one for the 17 1/2-inch diameter specimens and another for the 4-inch diameter specimens, so that the densities of specimens of these two sizes would be similar. Incorporation of the findings from the above and the principles of kneading-compaction appeared to show that in order to obtain adequate compaction of the proposed bituminous mixture by vibratory means, the aggregate in the mix had to be given plane motion, that is, both translation and rotation.

Proper compaction for the specimens of both sizes was achieved. The method employed made use of dynamic-impact, kneading action, and direct compression. For the compaction of the large specimens with the new testing machine, the following procedure was utilized:

1. A dead load of approximately 325 pounds was applied.
2. A dynamic force caused by the rotation of the eccentric masses of about 400 pounds at a frequency of 1200 revolutions per minute was used.
3. The molding head was canted (3/16 inch in 17 1/2 inches) and also oscillated continuously during the period of dynamic compaction through a horizontal angle of 40 degrees.
4. The duration of compaction was varied with the weight of the mixture, 3 minutes for each 10,000 grams of mix.
5. A final leveling load of 1000 pounds per square inch was applied and maintained for two minutes in order to square the faces of the specimen.

A photograph of the set-up for molding the large specimens is shown in Figure 6. The total dynamic force of compaction was not simply the addition of the dead load and centrifugal force (725 pounds). The



Compaction Assembly for Large Specimens

Figure 6

force due to translation of the dead load had to be considered and was dependent upon the amount and type of mixture.

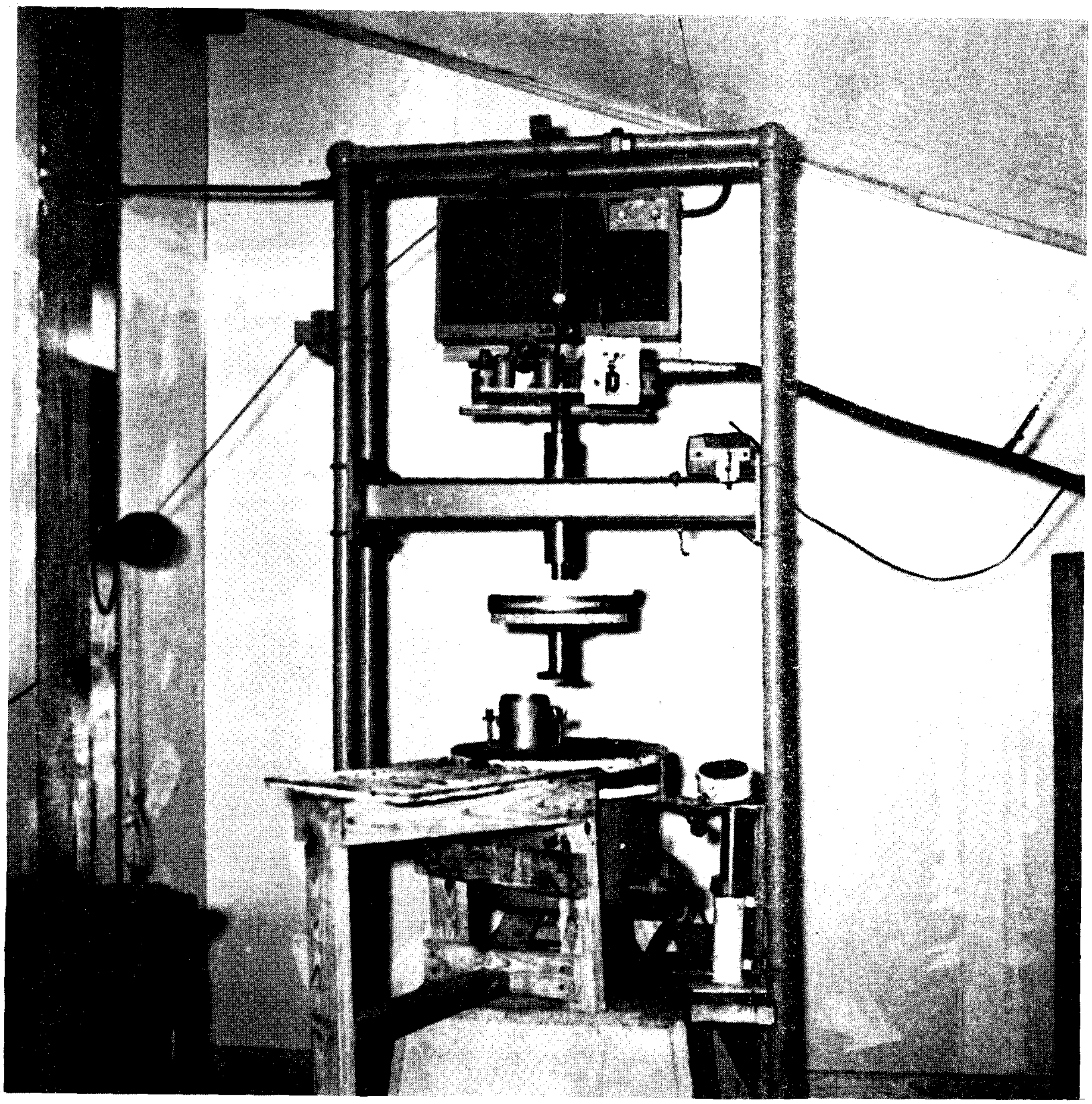
The compaction procedure and set-up (Figure 7) for the 4-inch specimens were similar to those described above with the following exceptions:

1. A dead load of 200 pounds was applied.
2. A dynamic force of 250 pounds at a frequency of 960 revolutions per minute was used.
3. The duration of vibratory molding was 2 minutes.
4. A final leveling load of 1000 pounds per square inch was applied and released immediately.

Detailed procedures for the compaction method used in this investigation are given in Appendix B.

Limited data were obtained on the comparison of specimen densities resulting from four methods used to compact asphaltic concrete in the laboratory. Table 1 shows such a comparison for one of the aggregate mixtures blended with different amounts and grades of asphalt. It can be seen that differences in amount of asphalt and grade of asphalt affected the densities of the samples compacted by all four methods and that the Texas Gyratory-Shear method generally resulted in higher densities.

Table 2 shows a sampling of data obtained for comparison of certain physical properties of specimens compacted by the Texas Gyratory-Shear and by the vibratory-kneading method used in this study. The Hveem Stability test was performed in accordance with the Texas Highway Department Method (42). The Hveem stability and cohesiometer values show that the particle orientation resulting from compaction differs for the two methods.



Compaction Assembly for Small Specimens

Figure 7

Table 1

Comparison of Specimen Densities Obtained by Various Methods of Compaction

	Texas Gyratory-Shear	Proposed Vibratory-Kneading	Marshall Method[2]	Immersion Compression[3]
Specimen Diameter, in.	4.00	4.00	4.00	4.00
Specimen Height, in.	2.00	2.00	2.50	4.00
Asphaltic Mixture	Density[1], gm/cc			
C-1-8.5-90[4]	2.349	2.361	2.320	2.302
C-1-7.5-65	2.295	2.210	2.234	2.205
C-1-6.5-90	2.228	2.115	2.188	2.193

[1] Average value for 3 specimens

[2] Standard of 50 blows on each face of specimen

[3] ASTM Designation D1074-52T

[4] Symbolizing compaction, aggregate mixture No. 1, 8.5% asphalt, by total weight, and asphalt grade OA-90 (85-100 penetration).

Table 2

Comparison of Physical Characteristics of Various Mixtures
Molded by Texas Gyratory-Shear Compaction
and Vibratory-Kneading Compaction
Determined by Hveem Stability and Cohesiometer

	Texas Gyratory 4"D Specimen			Vibratory-Kneading 4"D Specimen			Vibratory-Kneading 17 1/2"D Specimen
Asphaltic Mixture	Density gm/cc	Stab. %	Coh. gm/in.	Density gm/cc	Stab. %	Coh. gm/in.	Density gm/cc
R[1]-1-6.5-90	2.370	35	396	2.216	32	268	2.215
	2.364	37	338	2.229	32	253	2.209
	2.356	37	279	2.229	31.5	252	2.216
Avg.	2.363	36	321	2.225	32	258	2.213
R-1-7.1-90	2.364	32	382	2.253	28	330	2.231
	2.374	28	461	2.264	33	308	2.228
	2.375	26.5	408	2.267	33.5	280	2.232
Avg.	2.371	29	413	2.261	32	306	2.230
R-1-7.5-90	2.344	26	420	2.289	30	352	2.251
	2.363	24.5	455	2.292	30	318	2.278
	2.364	26	470	2.292	32	345	2.310
Avg.	2.357	26	448	2.291	31	339	2.280
R-1-8.0-90	2.356	18	320	2.343	25	540	2.297
	2.352	15	430	2.339	31	530	2.325
	2.349	15	450	2.339	31	540	2.298
Avg.	2.352	16	400	2.340	29	537	2.307

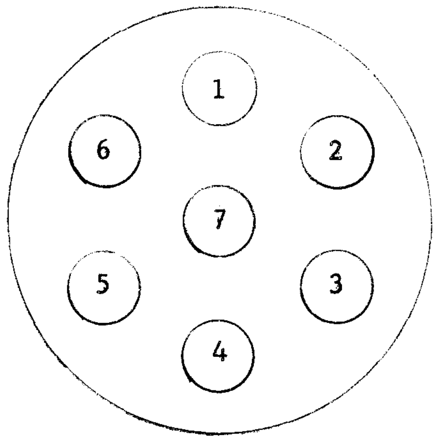
[1] The R stands for a mixture tested under repetitive loading.

The uniformity of the compacted mixture throughout several large specimens was checked by coring or cutting the specimens and then testing the pieces. Table 3 shows the variations obtained for density, stability, and cohesiometer value. The table shows that the range of stability or cohesiometer value was within acceptable limits; however, the range of density was somewhat high, but generally the individual pieces were about $\pm$ 0.03 gram per cubic centimeter from the average density. The data on density indicate that the cutting operation affected the pieces in that the average densities were higher than the original values. One of the cored specimens is shown in Figure 8.

The method of compaction described has been used in molding 4 and 17 1/2-inch diameter specimens from several asphaltic mixtures currently used for paving in the State of Texas.

Table 3

Uniformity of Compacted Large Specimens



Sample C-2-8.0-90	Density gm/cc	Height in.	Hveem Stability %	Cohesiometer gm/in. width
Total	2.282			
Core #1	2.269	1.69	18	302
2	2.279	1.67	21	265
3	2.272	1.66	21	270
4	2.277	1.66	20	290
5	2.323	1.67	22	315
6	2.274	1.70	20	270
7	2.310	1.70	21	250
Core Avg.	2.286	1.68	20	280
Remainder	2.284			

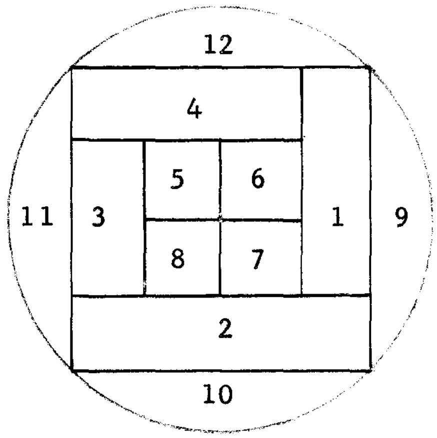
continued ...

Table 3 (cont'd)

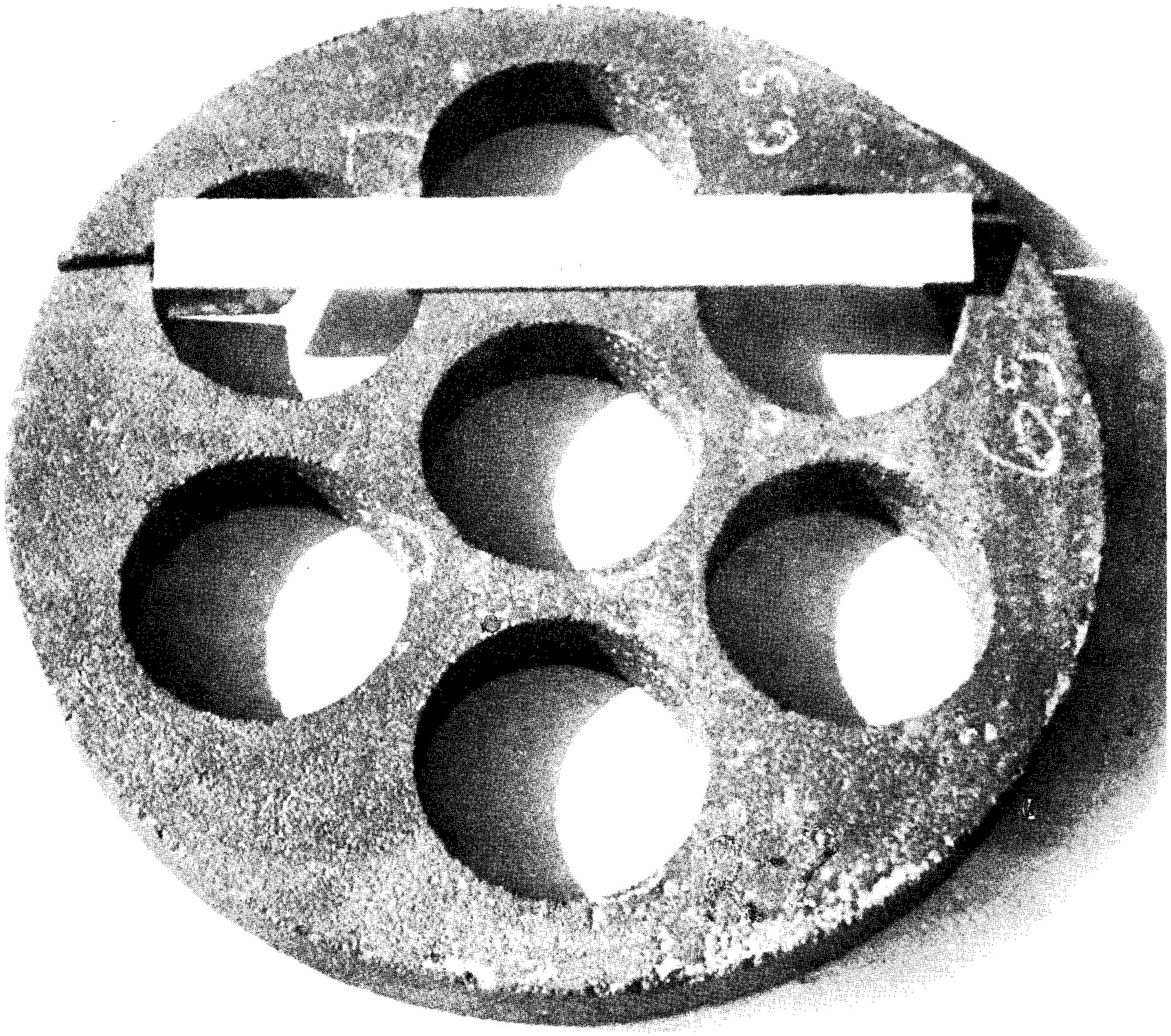
Sample C-2-6.5-90	Density	Height	Stability	Cohesiometer
Total	2.158			
Core #1	2.151	1.72	22	305
2	2.148	1.73	20	260
3	2.183	1.74	21	302
4	2.148	1.76	24	312
5	2.169	1.77	26	310
6	2.182	1.73	24	355
7	2.151	1.76	26	292
Core Avg.	2.162	1.74	23	305
Remainder	2.161			

continued ...

Table 3 (cont'd)



Sample C-2-7.5-90	Density
Total	2.260
Piece #1	2.261
2	2.258
3	2.261
4	2.282
5	2.277
6	2.262
7	2.274
8	2.306
9	2.257
10	2.278
11	2.242
12	2.251
Piece Avg.	2.267



Cored Specimens for Determination of Sample Uniformity

Figure 8

OUTLINE OF EVALUATION OF THE DEFLECTOMETER

A basic objective of this investigation was the evaluation of the deflectometer for testing asphaltic concrete under repeated flexures. In order for the device to serve this useful purpose, it must reasonably reproduce test data on similar specimens and must be sensitive to factors which are considered to affect the flexibility of asphaltic concrete. Several factors were included as variables in the testing program. These were divided into two parts; one concerns the loading of the specimen, and the other concerns the physical properties of the specimen. The data collected is extensive but not necessarily intensive; thus general or fundamental behavior will be established rather than specific deliniation on the resistance of asphaltic concrete to repetitive flexural deformations.

The design of the deflectometer is such that many variations in loading conditions can be obtained, such as, (a) speed of loading, (b) magnitude of load, (c) ratio of live load to dead load, and (d) support pressure. The particular loading variables investigated were load contact pressure and support pressure. Three different load contact pressures were obtained from one load by the use of loading discs of various diameters. Also, three different support pressures were utilized. Investigation of these different loading conditions was considered necessary in order to establish the deflection or load distribution patterns of the test specimens.

Numerous physical characteristics of asphaltic concrete can affect its behavior under repetitive loading. In the evaluation of the testing apparatus it was not deemed necessary to investigate all factors that were considered as being important. In fact, one of the variables (asphalt penetration) chosen for study may not be considered, by some, as affecting the flexibility of an asphaltic surfacing in service.

The specimen variables are listed below.

1. Asphalt content
2. Specimen thickness
3. Aggregate surface texture
4. Specimen density
5. Asphalt penetration

Limited information from the review of literature (2,16) indicates that the property of flexibility of asphaltic concrete is improved with an increase in asphalt content; but no limits on asphalt content were set forth with respect to this property. It has been shown that thin specimens (21) and thin asphaltic surfacings (1) are more flexible than thicker ones, but the variable of specimen thickness was included for the evaluation of the testing apparatus and the flexural formulae anticipated for use in reducing the test data. Aggregate surface texture was considered to be important for the property of flexibility as was specimen density. Whether or not the consistency of asphalt as determined by the penetration test plays an important role in the flexibility of an asphaltic concrete surfacing has been a controversial subject. The inclusion of this variable was made for the purpose of adding information on this subject.

COMPOSITION AND PROPERTIES OF ASPHALTIC MIXTURES

The asphaltic mixtures chosen for the major portion of this study were classified as coarse sheet-asphalt mixes. The aggregates used passed through a No. 4 sieve (0.187 in. openings). The following subsections describe in details the properties of the individual components and total mixtures.

Aggregates

Two different aggregate mixtures were utilized in the formulation of the coarse sheet-asphalt specimens, but the particle size distribution for both mixes was identical. One aggregate blend was obtained by combining a wet-bottom boiler slag and crushed limestone screenings; the other by combining a washed terrace sand with the same limestone screenings. A satisfactory proportion of the slag-limestone blend had been established by previous investigation which showed slag to limestone screenings weight ratio of 3:1. For the sake of blending economy the ratio suggested was altered slightly so that the aggregate blend used in this study was actually 76 percent slag and 24 percent limestone screenings. The sieve analysis and surface area of the combined materials are presented in Table 4 and the gradation curve in Figure 9.

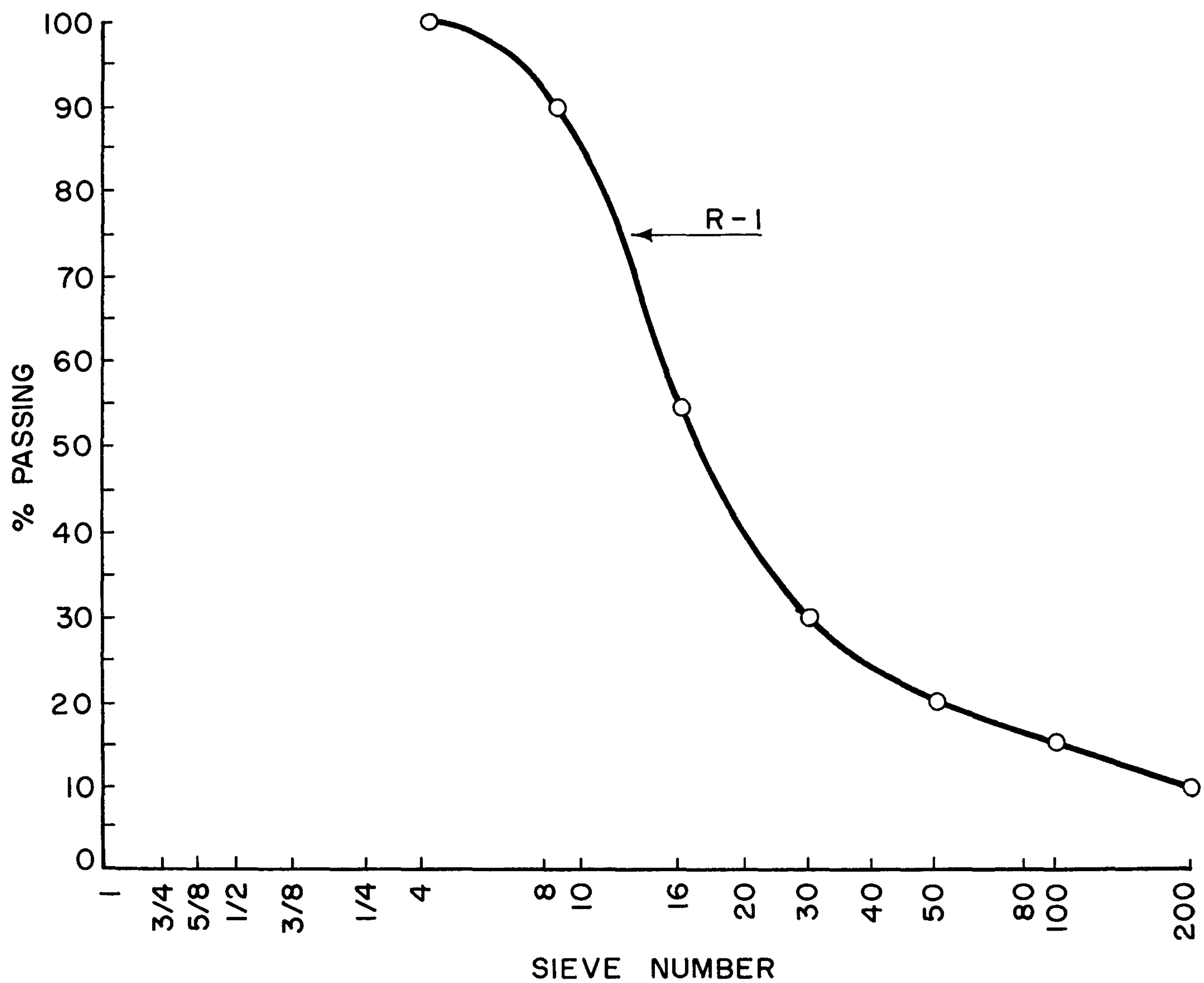
The wet-bottom boiler slag used as the primary aggregate in this research is procured as a by-product from the burning of lignite. This particular material is distinguished from other slags (dross) that are obtained from molten ore in that it is not crushed and its interstices are generally smaller. Asphaltic mixtures utilizing this slag and limestone screenings have been produced for overlays on portland cement

Table 4

Sieve Analysis of Coarse Sheet-Asphalt

Sieve	Total Percent Passing
4	100
8	90
16	55
30	30
50	20
100	15
200	10
Surface area[1], sq.ft./lb.	45.2

[1]Obtained by use of the California Highway Department surface area factors (43).



GRADATION CURVE FOR AGGREGATE BLENDS

Figure 9

concrete pavements and for direct applications onto prepared road bases. At asphalt contents of 6.5 to 7.5 percent, these mixtures have exhibited great resistance to reflection cracking and ability to flow without distress, even though they were placed in relatively thin layers, approximately 3/4 inch in thickness.

In a preceding section reference was made to the high internal friction of the asphalt-slag mixture. This resistance is brought about by the highly textured surface of the slag. In order to compare the relative effects of aggregate surface texture on resistance to repeated flexures, the fine-grained aggregate blend of siliceous sand and limestone screenings was included in this research. As mentioned previously, the gradations of the slag and the sand mixtures were identical, and the amount of limestone in each blend was constant.

The specific gravities and absorption values for these aggregates as determined by standard tests (ASTM C128-42) are presented in Table 5.

Proper control of aggregate gradation was obtained by separating the slag or sand into 5 different sizes and the limestone screenings into 4 sizes of particles, and then these were combined by weight to the desired proportions.

Asphaltic Cements

The asphaltic cements used in study of flexibility were obtained from one source. These were standard paving asphalts meeting the ASTM requirements for penetration grades of 120-150, 85-100, and 60-70, and the specifications of the Texas Highway Department (43) for the grades of OA-135 and 90. The Texas Highway Department does not list

Table 5

Physical Properties of Aggregates Used in
Sheet-Asphalt Mixtures

Aggregate	Bulk Sp.Gr. gm/cc	SSD Sp.Gr. gm/cc	Apparent Sp.Gr. gm/cc	Absorption %
Slag	2.564	2.656	2.826	3.61
Sand	2.612	2.632	2.663	0.75
Limestone Screenings	2.567	2.611	2.686	1.71

specifications for an asphalt grade of OA-65. The greatest use was made of the OA-90 asphalt. The standard penetration test values determined in accordance with the method of ASTM Designation D-5-52 are listed in Table 6. The absolute viscosities of these three grades of asphalt are also shown in Table 6. The viscosities at different temperatures were found by use of a sliding plate microfilm viscometer following procedures described by Griffin, Miles and Penther (42). These values are plotted in Figure 10 on logarithmic coordinates for viscosity and temperature. Since the temperature-viscosity relationship for the three asphalts is approximately linear in the double logarithmic plot and the three lines are practically parallel to each other, it appears that the asphalts had comparable susceptibilities to temperature. This assumption will be used later in the evaluation of results obtained from the testing program.

Preparation of Asphaltic Mixture

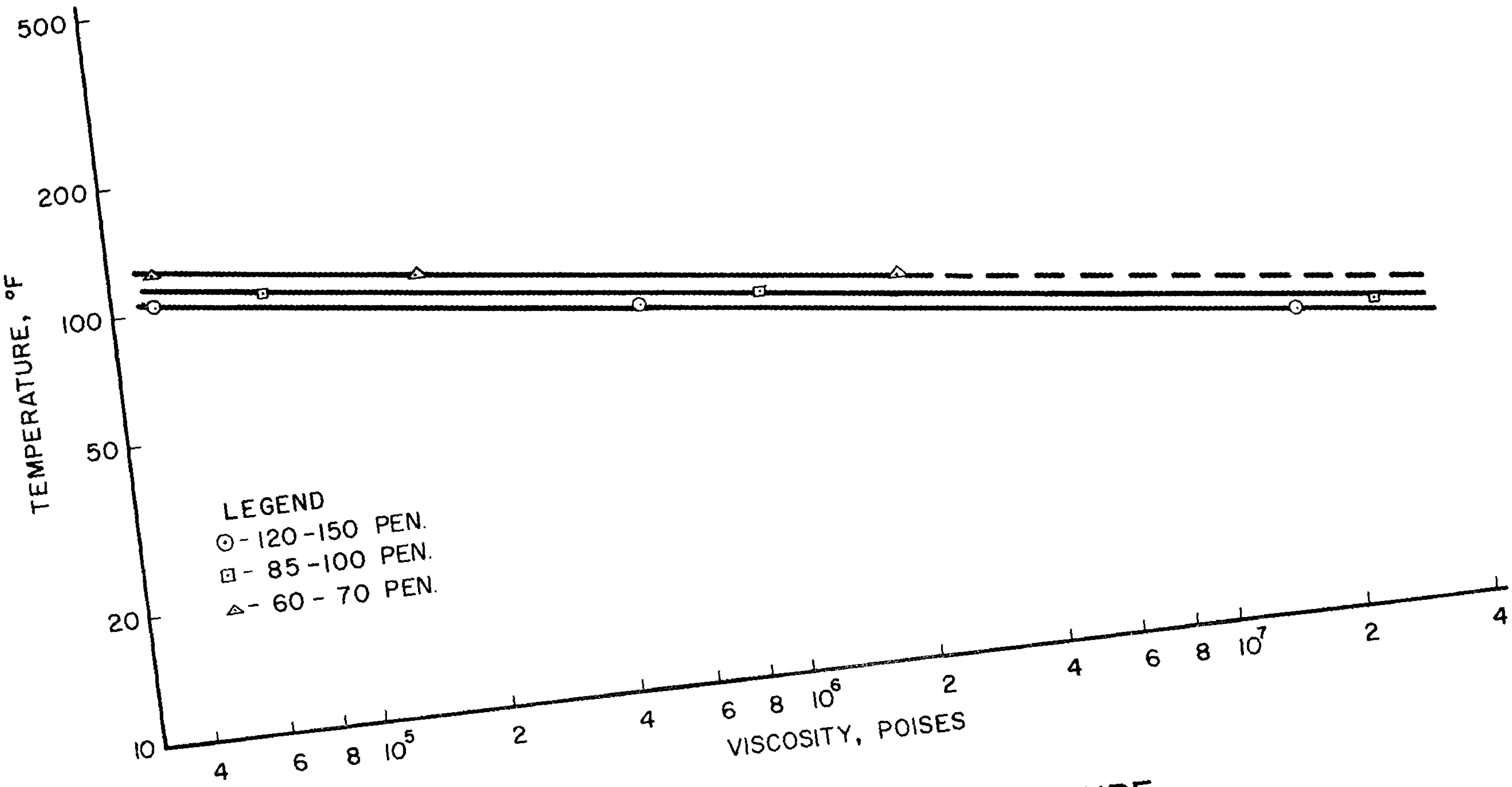
The procedure used in preparation of the mixes was one suited to the need of producing identical combinations of asphalt and aggregate. The previously separated aggregate sizes were combined by weight to the desired proportions and in sufficient quantities for one specimen. The aggregate was heated to a temperature of 300 $\pm$ 5°F in a forced-draft oven. The asphalt was heated to a temperature of 250 $\pm$ 5°F in a sealed metal container and in minimum amounts as required for the number of specimens to be batched. In no case was previously heated asphalt incorporated in a mixture to be used for testing.

The hot aggregate was transferred to a previously heated mixing bowl; the bowl was then tared on a weighing scale, and the proper amount

Table 6

Viscosity-Temperature Relationship of Asphalts by Sliding Plate Microfilm Viscometer

Designation ASTM	Designation Texas	Penetration 100gm/5sec/77°F	Specific Gravity 77/77°F	Temp. °F	Viscosity, Megapoises at shear rate of $5x10^{-2}sec^{-1}$
120-150	OA-135	122	1.016	50	16.5
				77	0.50
				104	0.038
85-100	OA-90	90	1.018	50	25.0
				77	0.96
				104	0.068
60-70	OA-65	60	1.020	77	2.00
				104	0.154
				122	0.038



ASPHALT VISCOSITY VS. TEMPERATURE

Figure 10

of asphalt was added to the aggregate. Mixing was accomplished with a Hobart C-10 food mixer in a period of at least two minutes. For the few cases that the specimen weight exceeded 10,000 grams (capacity of the mixing bowl), two smaller and equal batches were machine-mixed separately and then combined with further hand-mixing.

After mixing, the batch was placed in an open metal pan and stored in a 140°F forced-draft oven for a period of 15 hours prior to compaction or sampling for specific gravity determinations. It is common practice to cure laboratory asphaltic mixtures in order to allow for absorption of the asphalt by the aggregate.

Mixture-Design Properties

The physical properties of asphaltic mixtures described in this section pertain primarily to the mixes made with OA-90 asphalt. Data for the other mixes may be found in Appendix C, Tables 8 and 9.

After a mix had been cured for 15 hours at 140°F, representative portions were taken for the determinations of the impregnated specific gravity and the stability of molded samples. The impregnated specific gravity was found after the aerosol-vacuum method of Benson and Subbaraju (44). Gallaway (45) has indicated that this method of obtaining a specific gravity, from which the determination of voids is to be made, is "reliable and gives practical specific gravity values that consider most of the absorption of the asphalt by the aggregate."

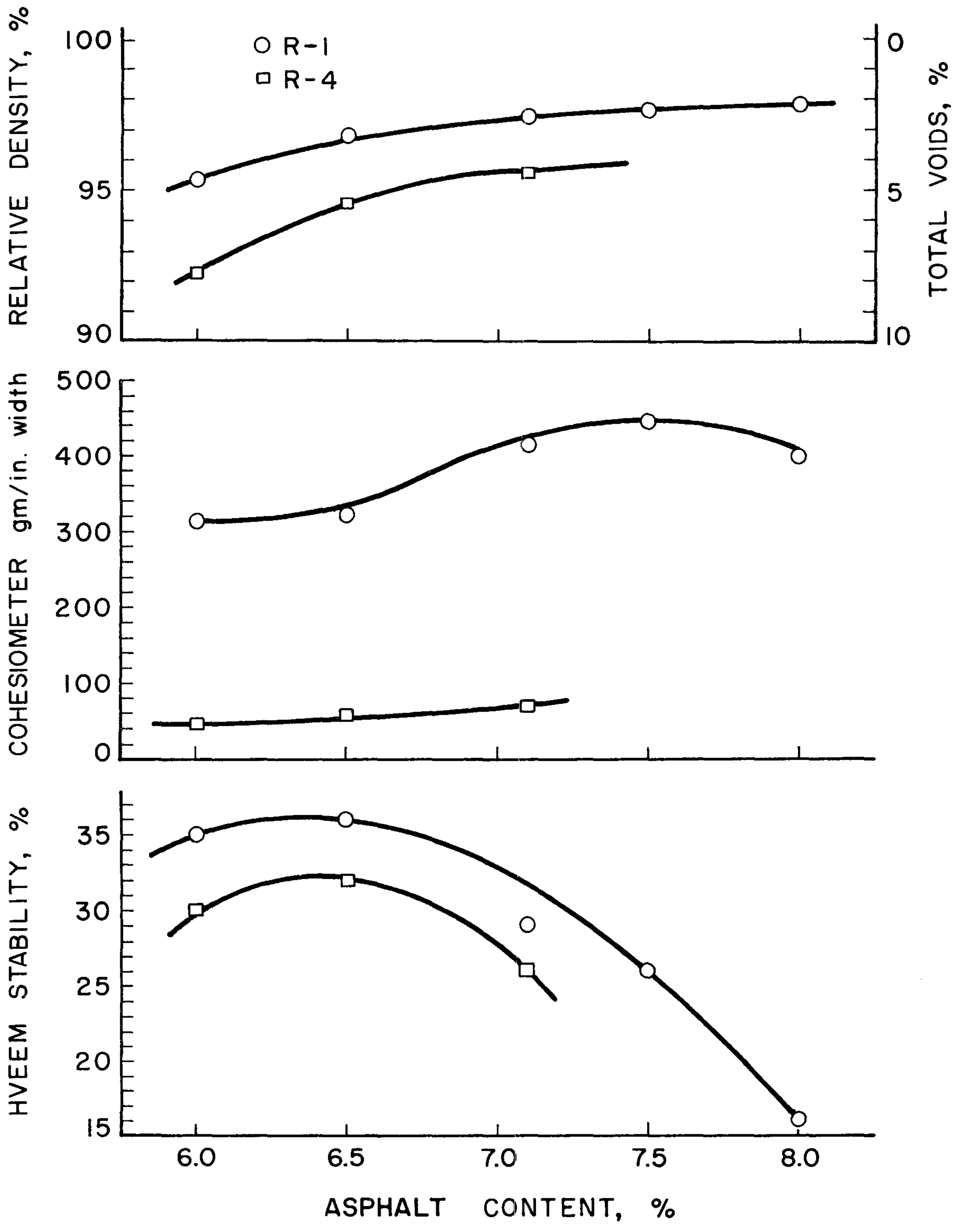
The samples of the mixes selected for strength purposes were molded (47) and tested (40) in accordance with methods of the Texas Highway Department. The design values of these mixtures are tabulated in

Table 7 and are also presented in Figure 11. The asphalt content is defined in terms of percent by weight of total mixture. R-1 mixes contained slag and R-4 combinations had siliceous sand.

Table 7

Design Values of R-1 and R-4 Mixtures with 85-100 Penetration
Molded by the Texas Gyratory-Shear Method

Asphalt Content %	Specimen Density gm/cc	Impregnated Sp.Gr.	Relative Density %	Total Voids %	Hveem Stability %	Cohesiometer value gm/in.
			R-1-90 Mixture			
6.0	2.341	2.455	95.3	4.7	35	313
6.5	2.363	2.440	96.8	3.2	36	321
7.1	2.371	2.435	97.4	2.6	29	413
7.5	2.357	2.416	97.6	2.4	26	448
8.0	2.352	2.405	97.8	2.2	16	400
			R-4-90 Mixture			
6.0	2.225	2.410	92.3	7.7	30	49
6.5	2.244	2.375	94.5	5.4	32	60
7.1	2.253	2.356	95.6	4.4	26	70



DESIGN CURVES FOR R-1-90 AND R-4-90

Figure 11

REPETITIVE LOAD TEST PROCEDURE

In preparation for testing of a specimen, certain techniques had to be evolved in order to duplicate results obtained for the specimens constituting a set. All specimens were kept at an air temperature of $75 \pm 2^{o}F$ for a minimum period of seven days prior to testing. The following steps describe the method used in preparing a sample for testing, for loading, and dismounting of the specimen:

1. The reaction unit was checked for "calibration" of air (Appendix A). Before the steel plate specimen was removed the oil pressure in the system was set at approximately 0.05 psi and then the valve between the pump and the oil chamber was closed.
2. A 17 1/2-inch diameter paper disc with radial slits was secured to the rubber membrane with masking tape.
3. The specimen was then centered on the reaction unit and a rubber band of proper size was stretched around the circumference of the specimen.
4. The clamping ring and bolt assemblies were placed to secure the specimen onto the reaction unit. The bolts were drawn in such a manner to apply the clamping force as uniformly as possible around the periphery of the specimen. When the oil pressure within the unit increased during the clamping operation, it was reduced to the original setting. The bolts were also tightened sufficiently to bring the spacer sleeve into contact with the shoulder of the oil chamber.

5. The dial carriage was secured to the reaction unit and dial gages placed.
6. The loading disc of the loading system was brought nearly to the surface of the specimen, and then the unit was fixed to prevent rotation of the disc but not translation.
7. All dial gages were zeroed. The load-disc dial gage attached to the bottom proving ring plate was zeroed when the loading disc made slight contact with the face of the specimen.
8. The dead load was applied by releasing the elevating cable, and immediately the desired supporting pressure to the specimen was impressed; then the electric motor energizing the live load was started. The load-disc dial gage was read and recorded after releasing the dead load and after application of the supporting pressure.
9. At various intervals of time the pressure and dial gages were read and recorded, and a running plot was kept of the load-disc dial reading versus number of load repetitions on logarithmic coordinates.
10. After initial adjustment of the specimen to load, the bolts on the clamping ring were checked and tightened if necessary.
11. The loading of the specimen was continued, and a record of the load-disc dial gage reading was kept until the plot of central deflection versus number of load application deviated for at least three consecutive points from a straight line established by previous readings.

12. At the end of the test the loading motor was stopped and the loading system was elevated from the specimen and secured. The pressure acting on the bottom of the specimen was relieved, and then the dial carriage was removed. Upon release from the reaction unit, the specimen was examined for cracks and these were marked with a crayon. The failed specimens were preserved for final density determinations.

A number of comments are pertinent to the testing procedure described. The sequence of applying the dead load first and then the supporting pressure was necessary since for a thin flexible specimen, it was impossible to raise the oil pressure to one pound per square inch due to the limited volume of oil in the jack and to the deformability of the specimen. Of course, it may have been desirable to have reached the maximum values of the two loads simultaneously, but this procedure would not have precluded the deformation of the specimen prior to adding the live load.

The rubber band placed around the specimens was utilized to give it some lateral restraint of an indeterminate amount so as to minimize the circumferential flow of the mixture and thus forestall the need of continuous tightening of the clamping bolts. It was found that the band was needed only for the specimens with high asphalt content.

The reading of the dial gages was performed by the operator. Early trials in using high speed photography for recording the dial readings show no benefits over the method of reading the dials by "eye" for the frequencies of load applications investigated.

Considering the load frequency of 11 repetitions per second, the greatest film speed of 64 frames per second employed was not satisfactory for determining the maximum and minimum dial readings.

Consideration was given to the effect that the acceleration given a dial stem would have on the value of movement indicated by the dial. Comparisons were made for the peak values of readings obtained while the indicator needle was in motion with the peak values read when the stem of the dial was held stationary. It was found that for 2 1/4-inch face dials, inertial effects on the peak value read were not significant for amplitudes of deflections less than about 0.030 inch. For the cases where this limiting amplitude occurred, the peak dial readings were established by manually supporting the dial stem so that rotation of the needle would be eliminated during the period of reading the dial.

Use of the proving ring during testing was not a real necessity for the determination of the maximum loads imposed on the specimens. These could be evaluated from knowledge of the dead load, of the maximum force exerted by the rotation of the eccentrics, and of the distance the dead load was translated. The proving ring was kept in the loading system due to custom observed in earlier studies.

Loadings of Repetitive Tests

Experimentation with the deflectometer demonstrated its versatility in the study of numerous loading variables individually or in combinations including the following.

1. Frequency and rate of loading
2. Magnitude of load

3. Ratio of dead load to live load

4. Loading-disc area

5. Initial support given the specimen

Of the different load conditions possible, the following was chosen for a standard.

1. Load frequency of 11 cycles per second

2. Dead load of 139 pounds

3. Live load due to rotation of eccentrics 104 pounds

4. Load contact area of 5.0 square inches

5. Initial support to specimen of 1.0 pounds per square inch

Particular consideration was given to the <u>standard</u> load conditions. The loading time was within the range (1/4 - 1/100 sec) stated by Nijboer and van der Poel (13) for duration of loads which a road endures under traffic and was near the lower frequency employed by Saal and Pell (23). The dead load of 139 pounds and live load of 104 pounds allowed a tolerance of 35 pounds for the force due to acceleration of the dead load in order to prevent impact. Loading the specimen over a central area of 5.0 square inches, yielded an average nominal pressure approximating 50 pounds per square inch. The choice of employing a one pound support pressure was determined from experience with the testing machine.

An example of the data sheet utilized for recording of the deformations indicated by the various dials is illustrated in Figure 12. The data sheet shows information concerning the specimen, loading conditions, and test air temperature. All tests were performed in a temperature controlled room. The variation of temperature from that desired ($75^{\circ}F$) was $\pm$ $2^{\circ}F$ when the thermometer was suspended in air.

DEFLECTOMETER DATA SHEET

Mix__________ Asp. Cont.________% Thick.________in. D.L.______# L.L.±______# Mass C,____ Date:
Molded________
Speed______rpm. Init.Press.______psi. Cont. Area______sq.in. Temp.______°F Spec.____________ Tested________

Density	Counter	Reps.	6" L		4" L		2 1/4"C		Load	4" R		6" R		Gage	Ring	Remarks
			D	d	D	d	D	d	Disc	D	d	D	d			
Before Test																
Wt.(air)______																
Wt.(H_2O)______																
Vol.,cc.______																
Sp.Gr.________																
After Test																
Wt.(air)______																
Wt.(H_2O)______																
Vol.,cc.______																
Sp.Gr.________																

Figure 12.

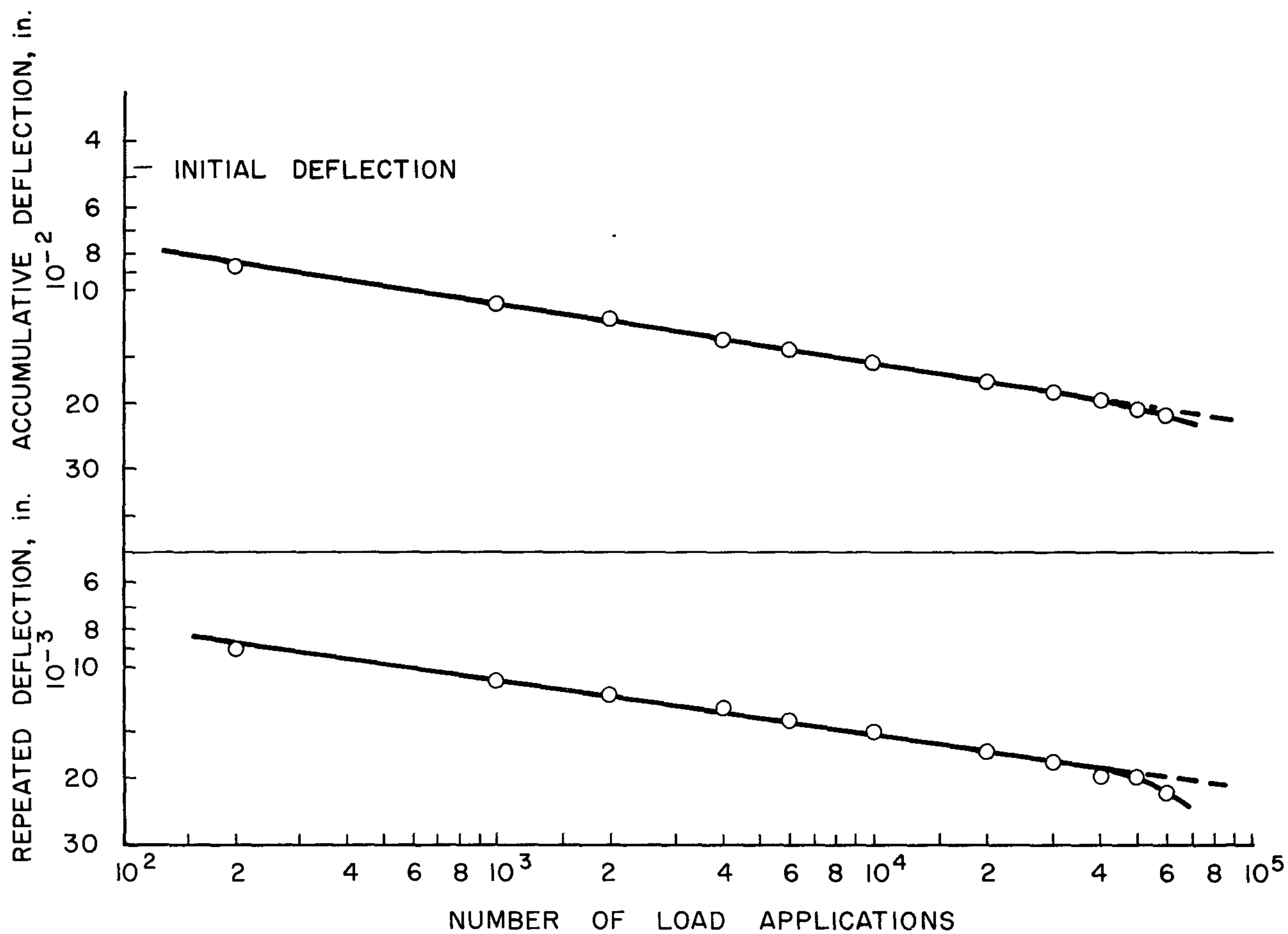
Determination of temperature within a specimen showed differences of less than $\pm$ 1^{o}F from 75^{o}F.

The criterion for failure of a specimen undergoing repeated flexures was established after noting the behavior prior to achieving complete rupture of the specimen. This behavior was revealed by the direction of the dials showing specimen deflections, load, and support pressure. Generally, failure had occurred when, after a certain number of load applications, one or a combination of the following changes became evident:

1. The support pressure increased to a maximum value and then decreased.
2. A dial gage indicated that the deflection of a point on the surface had reversed in direction.
3. The movement of the load indicating dial became erratic.
4. The plot of the loading disc (foot) deflection versus number of load repetitions on logarithmic coordinates deviated from a straight line.

Evidence of failure indicated by items 1, 2, and 3 always followed item 4. Repeated examinations of specimens satisfying the failure criterion of item 4 showed cracks on the bottom face of the specimens. For this reason the failure criterion of item 4 was accepted even though the specimen was generally still capable of resisting many more load applications before rupturing completely by punching shear.

Figure 13 presents a plot of the load-disc deflections versus number of load applications for a specimen of mix R-1-7.5-90. This figure shows two amounts of deflections, one the accumulated deflection and the other the repeated deflection. In general, the load-disc



LOAD-DISC DEFLECTION VS. NUMBER OF LOAD APPLICATIONS
R-1-7.5-90 STANDARD

Figure 13

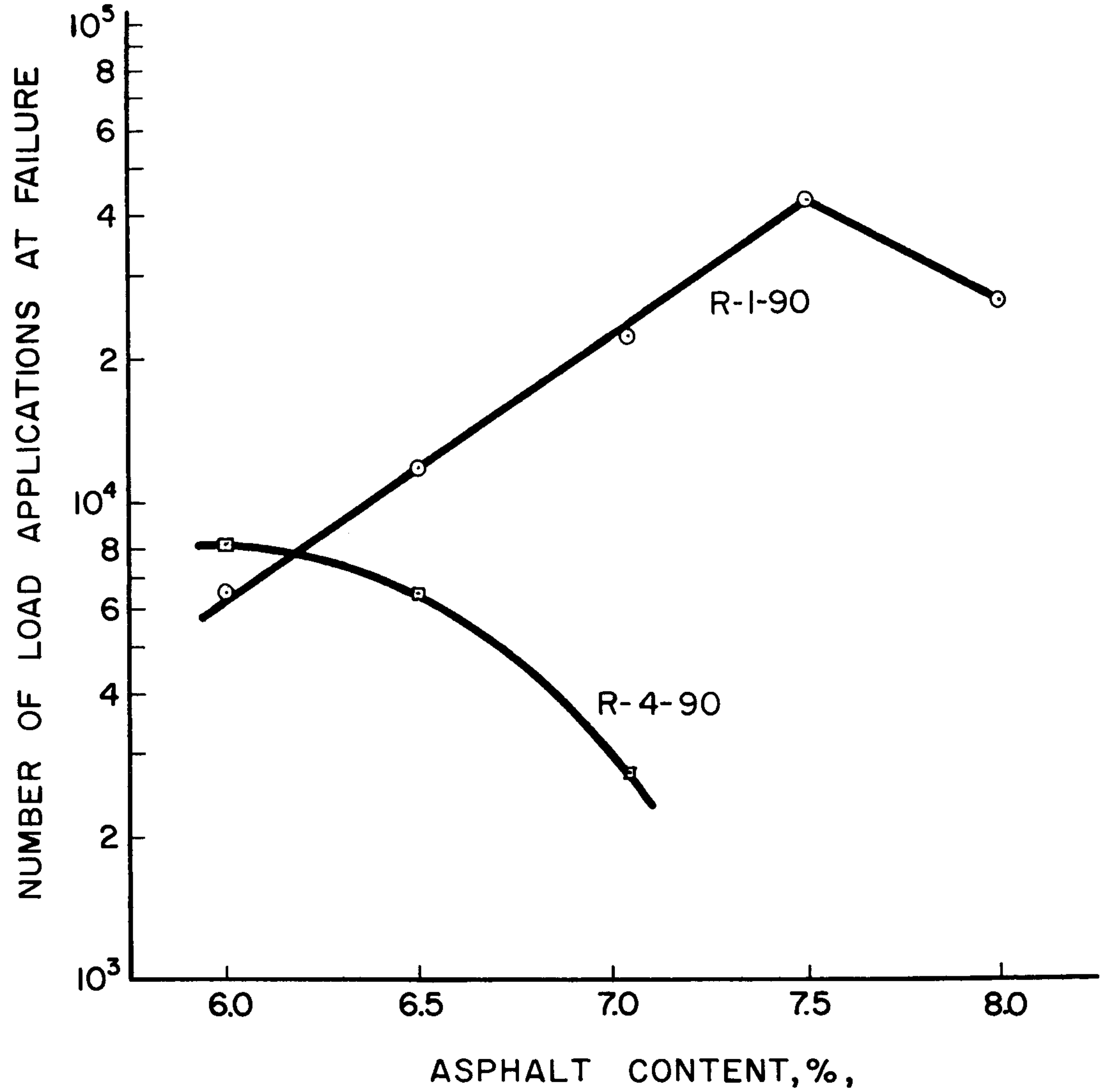
deflections recorded within the first 1,000 repetitions (1 1/2 minutes) were disregarded in establishing the straight lines since these were subject to variabilities of initial disc location and seating and also adjustment of the specimen to load.

TEST RESULTS AND DISCUSSION

This section contains the results of repetitive load tests performed on laboratory compacted asphaltic concrete specimens and also a discussion of the individual factors investigated which affected the resistance of the specimens to the imposed loads. In the sub-divisions of this section are discussed the effects of the following variables: (a) asphalt content, (b) specimen thickness, (c) load contact area, (d) initial specimen support, (e) asphalt consistency, (f) aggregate surface texture, and (g) specimen density. Other results obtained from this investigation which concern specimen deflection and the use of the Grashof equations will be discussed.

Asphalt Content

For most present methods of asphaltic concrete design, the stability of a mixture increases with an increase of asphalt content up to a limiting value of asphalt content, and then the stability decreases with further increase in the amount of asphalt. The review of literature has not shown whether similar behavior occurs for the relationship between asphalt content and resistance to repetitive loads of asphaltic concrete. The curves presented in the semi-logarithmic plot of Figure 14 indicate that such a relationship did exist for the two mixtures tested. Although the curve for the R-4 mix does not show a peaking effect, it can be rationalized that at some value of asphalt content less than 6.0 percent, the number of load repetitions to result in failure is less than the corresponding value at 6.0 percent asphalt



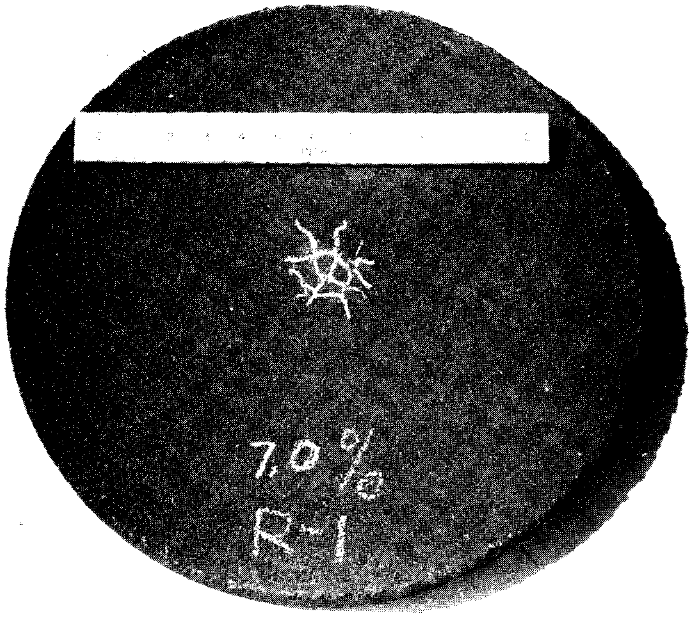
ASPHALT CONTENT VS.
NUMBER OF LOAD APPLICATIONS
AT FAILURE. STANDARD

Figure 14

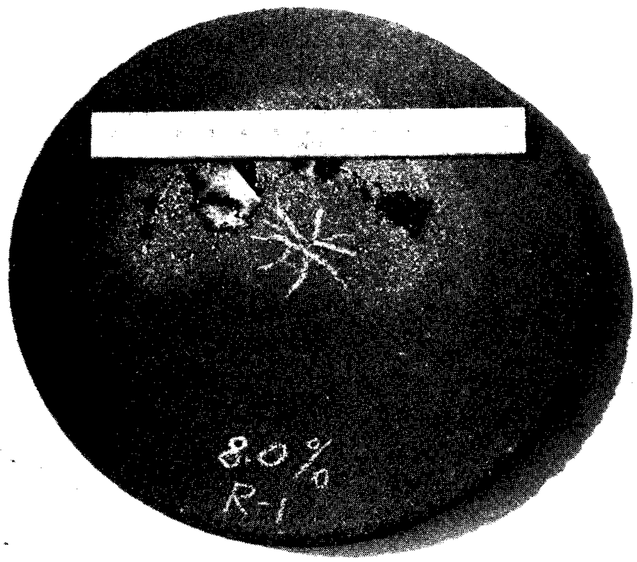
content. The reason for this behavior is not readily evident, but it can be conjectured that the effective asphalt film thickness is excessive and consequently reduces the frictional resistance between the aggregate particles.

The concept of asphalt film thickness as a basis for the design of asphaltic concrete has been presented by Campen, et al (48). The authors state that "....film thicknesses of 6 to 8 microns produce the most desirable pavement mixtures." The film thicknesses presented in Appendix C, Table 8, are actually ratios of the volume of asphalt divided by the computed surface area of the aggregate. This definition of film thickness, which assumes that the asphalt coats each particle to the same extent, may not be exactly true. The use of asphalt film thickness for the design of asphaltic concrete is interesting, but possibly the film thickness should be computed from the volume of asphalt in excess of that required to fill the surface voids of the aggregate. Figure 14 shows optimum asphalt contents of 7.5 percent for the R-1-90 mixture and approximately 6.0 percent for the R-4-90. At these asphalt contents, the R-1 blend had a computed film thickness of 8.6 microns, and for the R-4 mix it was 6.8 microns.

The photographs of Figure 15 show typical crack patterns produced on the bottom faces of R-1 specimens. Figure 15a shows a pattern similar to that found in asphaltic concrete surfacings in service (compare with Figure 1) in that the cracks are connected to form polygons. The photograph, Figure 15b, shows radial cracks that join at the center but are otherwise disconnected. This latter type of cracking may indicate that the asphalt film thickness was excessive since it is



(a)



(b)

Crack Patterns

Figure 15

comparable to disconnected cracks found aften in asphaltic surface treatments in which the asphalt performs in thick films. Only the crack pattern of Figure 15a was found for the R-4 mixtures.

Specimen Thickness

As was expected, the number of load applications to cause failure increased as the thickness of specimen increased. This behavior is illustrated by Figure 16 which presents a logarithmic plot of specimen thickness versus the number of load applications to cause failure. The curve is a straight line which can be expressed in the form:

$$y = bx^{m} \tag{6}$$

where y = thickness of specimen

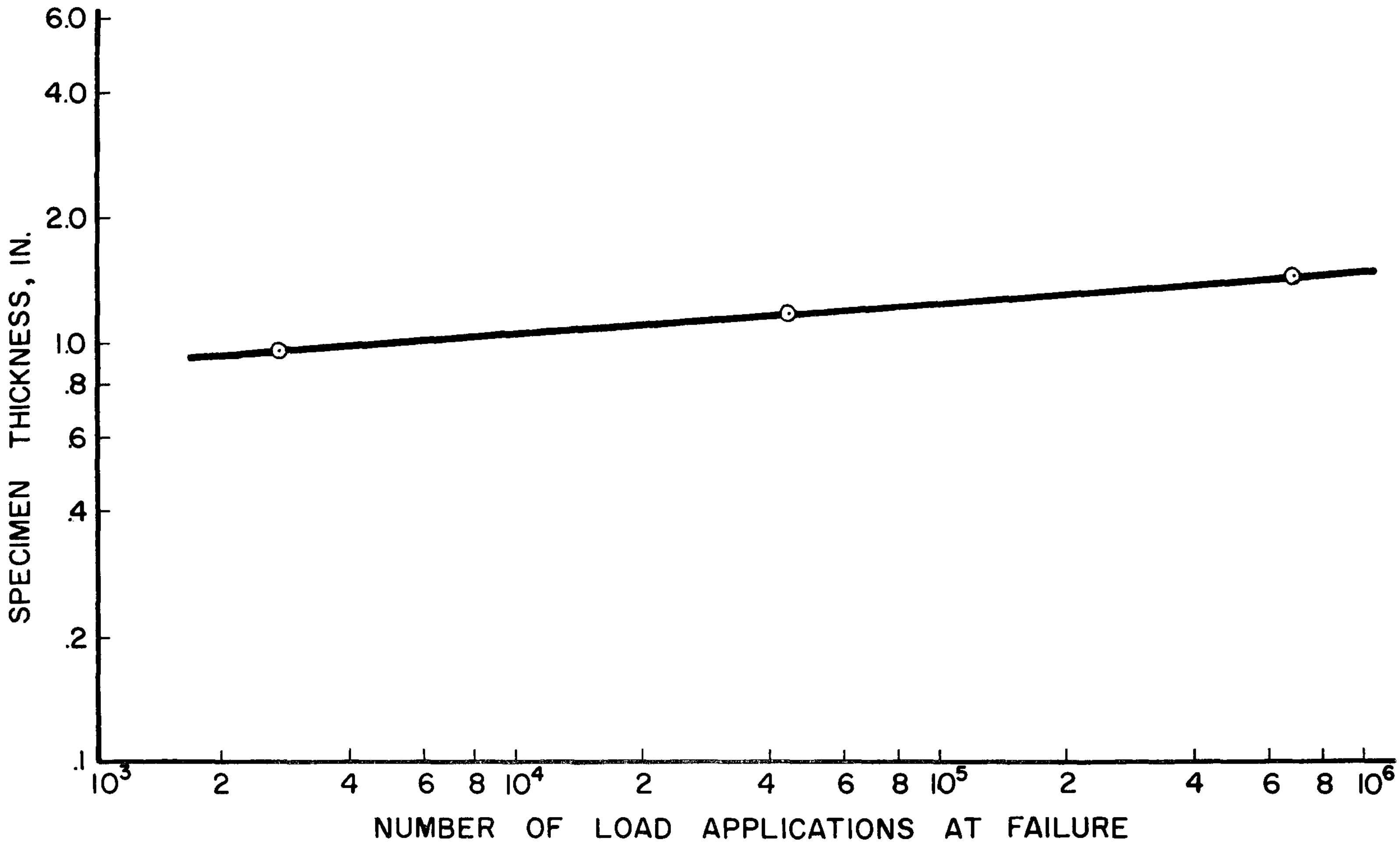
x = number of load application at failure

b and m = constants.

The constants $\underline{b}$ and $\underline{m}$ will be of different values for different asphaltic mixtures. These data indicate that there is a definite relationship between the thickness of specimen and its resistance to repetitive loads.

Load-Disc Area

Comparisons made to determine the effects of load contact area on the resistance of a mixture to the repeated load tests did not yield a simple relationship. The number of load applications at failure was 32,000 for the load-disc 3.14 square inches in area, 43,000 for the 5.00 square inch disc, and 138,000 for the 8.00 square inch loading disc. The trend is as might be expected. Failure to obtain a simple relationship between load contact area and resistance to the repetitive loads



SPECIMEN THICKNESS VS. LOAD APPLICATIONS AT FAILURE. R-1-7.5-90 STANDARD

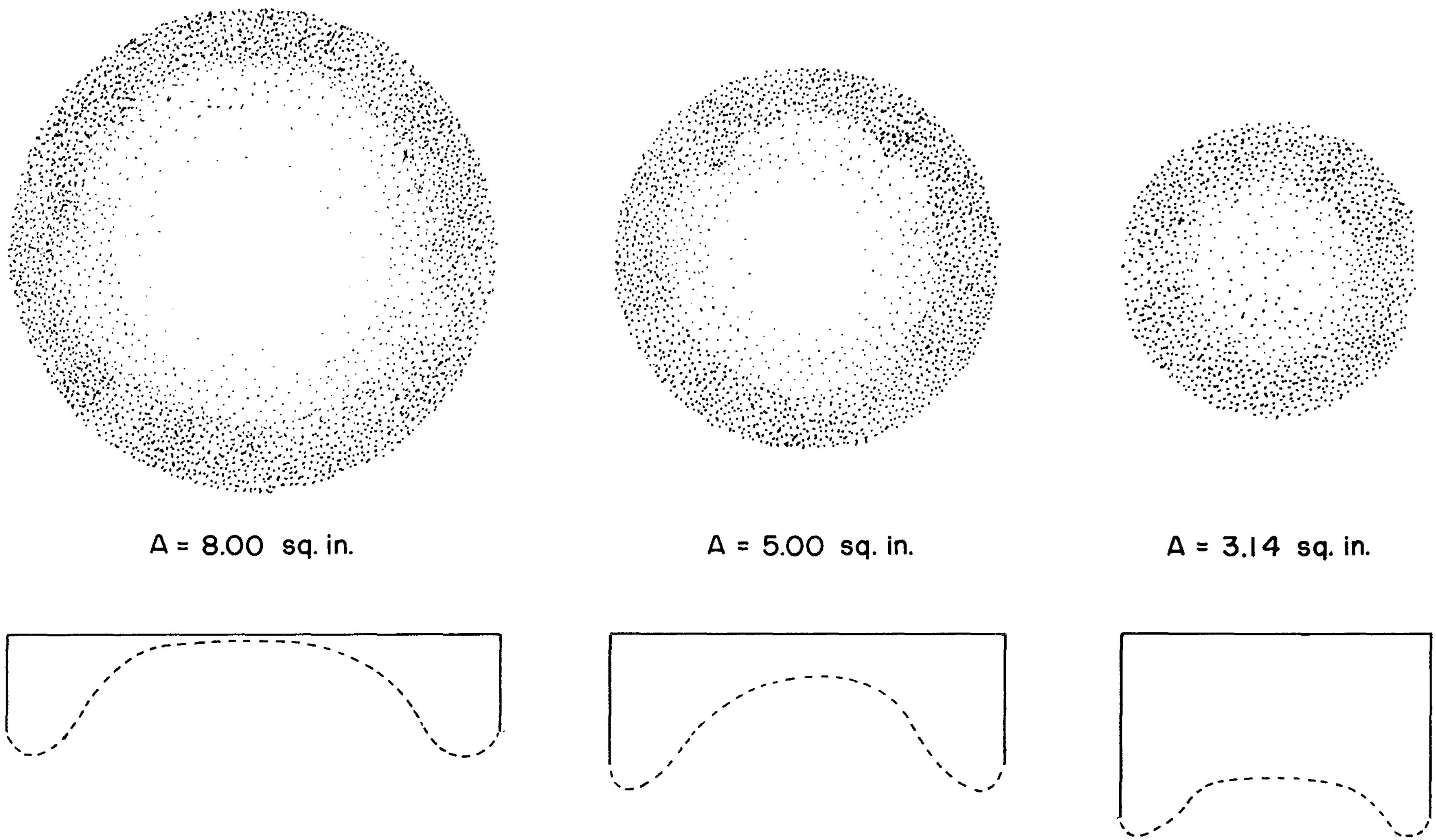
Figure 16

applied may be due to the limited amount of data obtained; but it is thought that differences in the distribution of load pressure on the contact surfaces are also responsible for this complex behavior. A pattern of pressure distribution intensity was obtained by placing a sandwiched carbon paper between the load-disc and a specimen and then loading the specimen. The patterns obtained for the three load-discs have been reproduced graphically in Figure 17; they show definite differences as to pressure distribution over the load surface. In examining these patterns one must realize that a minimum amount of pressure is required to cause a carbon imprint and that the blank central areas of the two larger discs do not imply that these areas were not directly stressed.

Initial Specimen Support

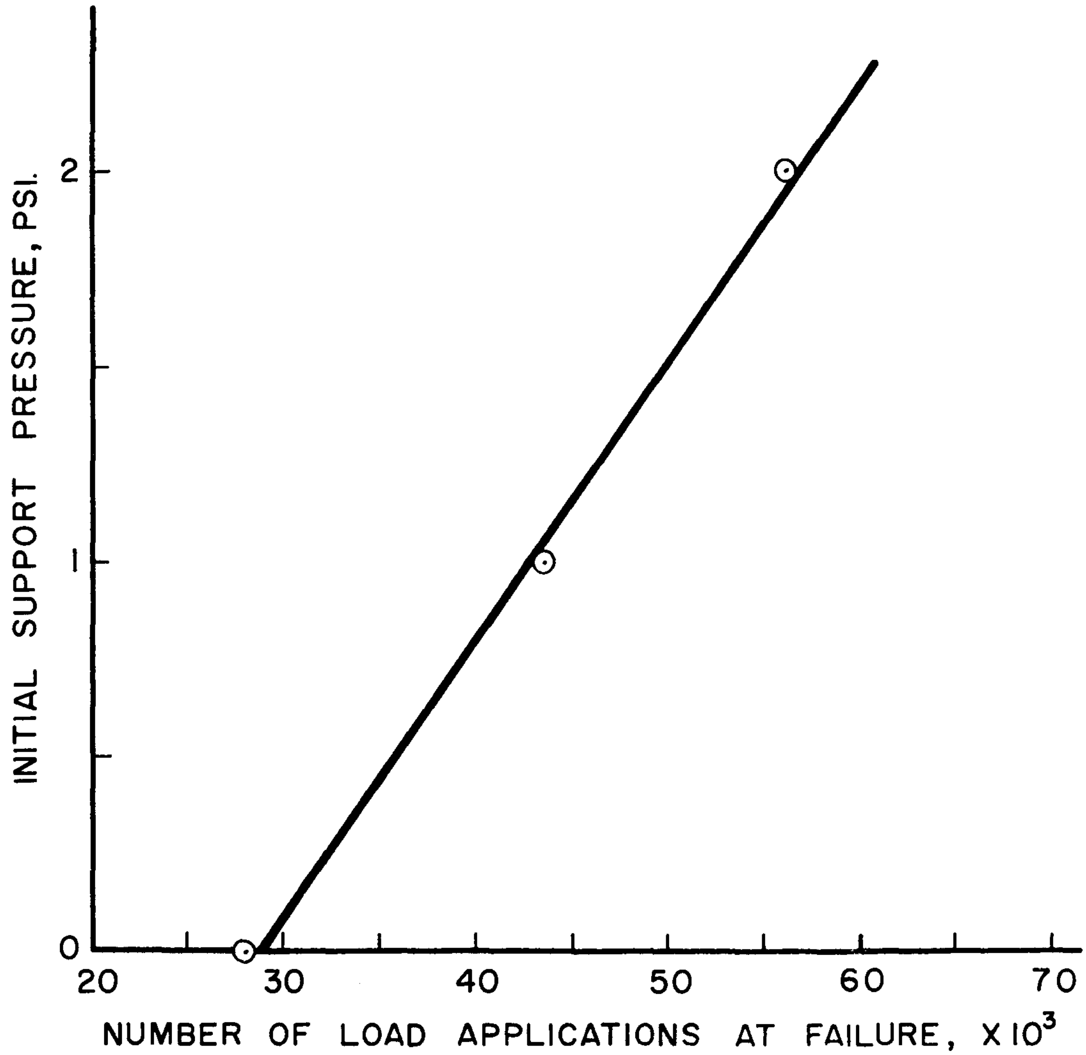
The support given any test sample will affect the load carrying capacity of the specimen. This particular condition was investigated to evaluate both the test apparatus and its effect on the measured strength of a specimen. The curve of Figure 18, showing a relationship between initial support and number of load applications at failure of similar specimens indicates that a one pound pressure increase of initial support increases the number of load applications to cause failure by nearly 15,000 repetitions. This illustration confirms the theory which states that the stresses induced on the test specimens are dependent on the initial support given the specimen.

Also of interest is the amount of accumulated load-disc deflection at failure. Figure 19 shows a simple linear relationship between



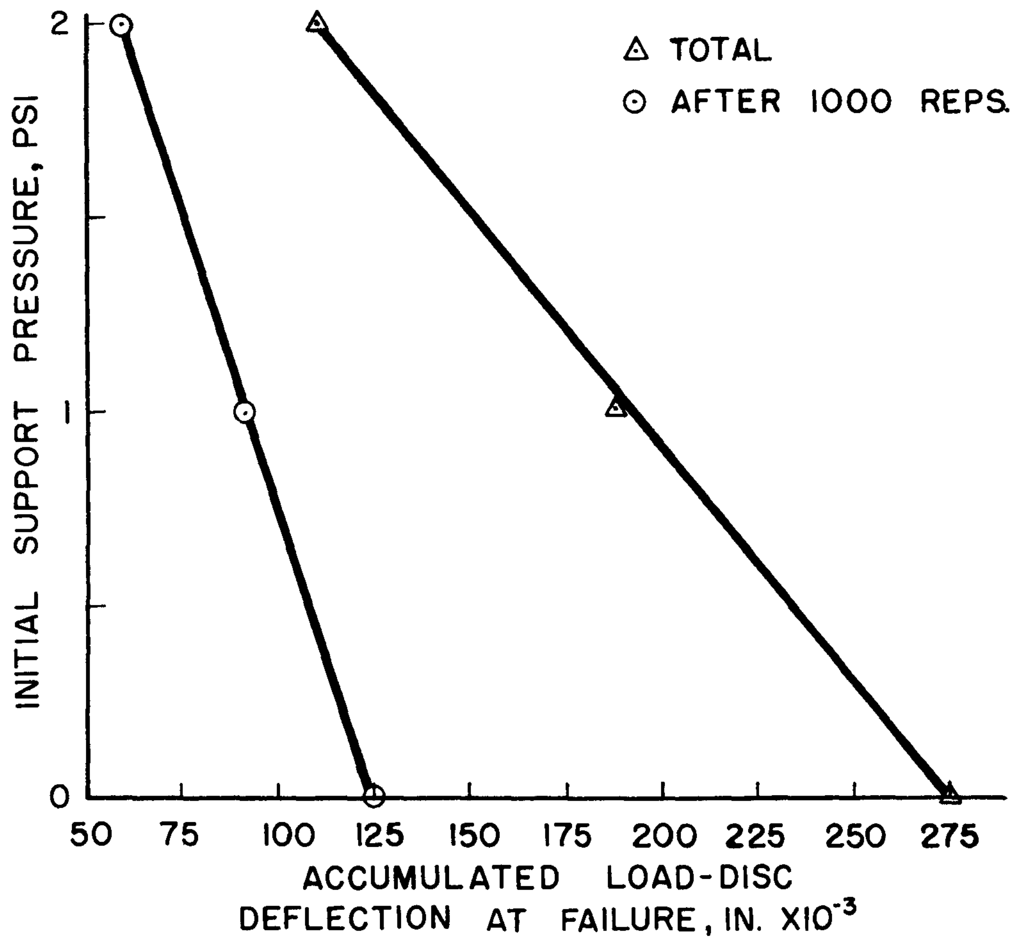
LOADING DISC IMPRINTS AND INDICATED PRESSURE DISTRIBUTION OF LOAD

Figure 17



INITIAL SUPPORT VS. NUMBER OF LOAD APPLICATIONS AT FAILURE
R-1-7.5-90 5.00 SQ. IN. DISC

Figure 18



INITIAL SUPPORT VS. ACCUMULATED LOAD-DISC DEFLECTION AT FAILURE R-1-7.5-90 5.00 SQ. IN. DISC

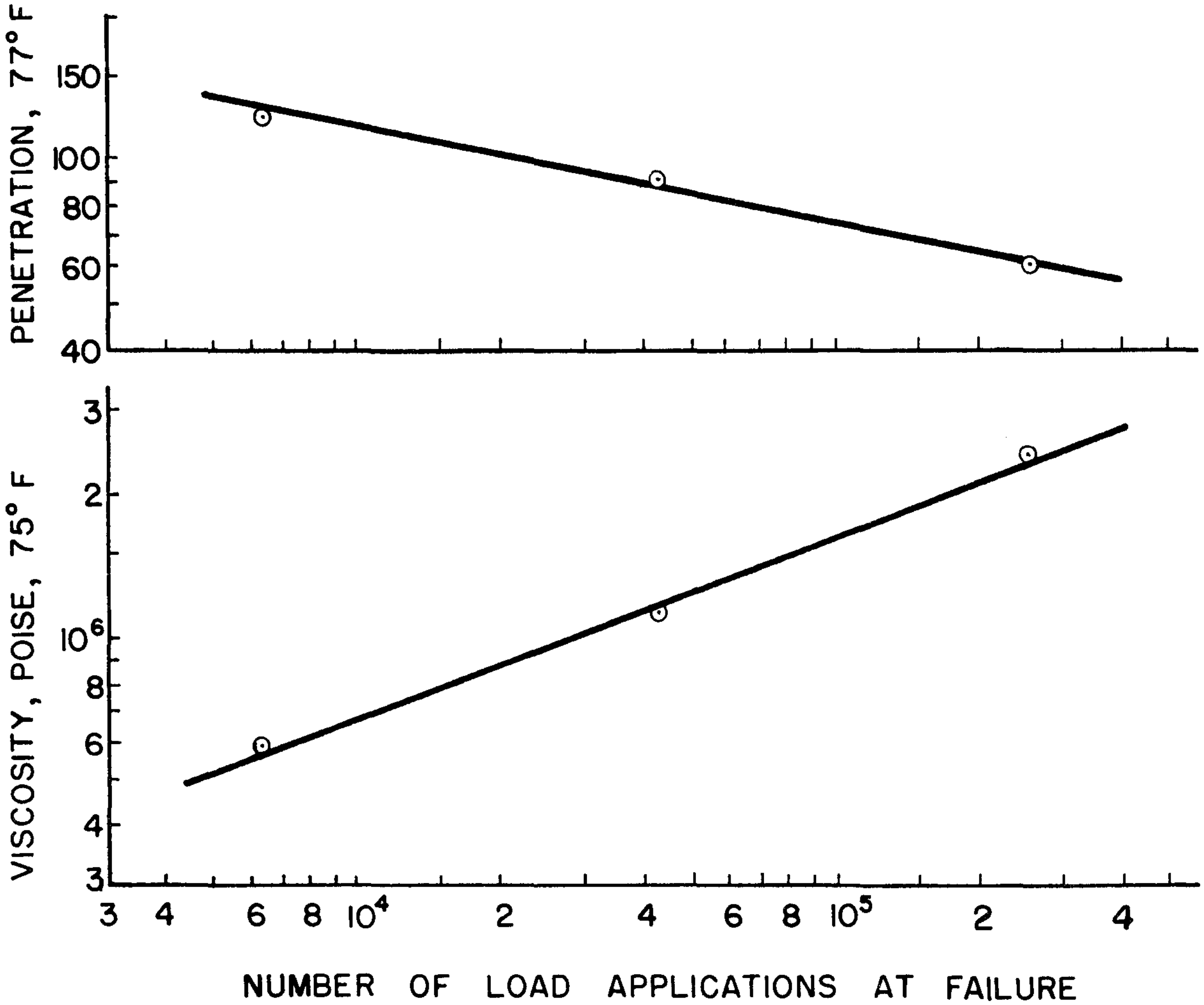
Figure 19

initial support and accumulated deflection at failure. As can be seen from Figures 18 and 19, the greater the initial support the smaller will be the amount of accumulated deflection at failure, and the greater will be the number of load applications to cause failure. The above findings verify the knowledge that endurance of a material to repeated loads is greater for the loads causing the lesser strains in the specimen. In this comparison specimen deflection is related to strain.

Asphalt Consistency

The logarithmic curves of Figure 20 compare asphalt consistency to the resistance of a specimen to repetitive loads and indicate that the more viscous the asphalt the greater is the number of load applications to result in failure. It has been implied that the amount of specimen deflection at failure should be considered in determining the flexibility characteristics of an asphaltic concrete mixture. The data from Appendix C, Table 11, show that the accumulative deflections of the specimen made with 7.5 percent of 120-150, 85-100, and 60-70 penetration asphalts were 0.240, 0.200, and 0.182 inch, respectively. The differences in deflections do show that the mixtures with softer asphalt can tolerate greater deflections before failure, but yet are less resistant to repeated loads.

As was shown in Figure 10, the temperature-viscosity curves for the three grades of asphalt were approximately straight lines and parallel to each other. From these curves the temperatures were found, for the 85-100 penetration asphalt, at which it would have the viscosities of the two other asphalts at the temperature of 75^{o}F. Substituting the



ASPHALT VISCOSITY AND PENETRATION VS. NUMBER OF LOAD APPLICATIONS AT FAILURE. R-1-7.5 STANDARD

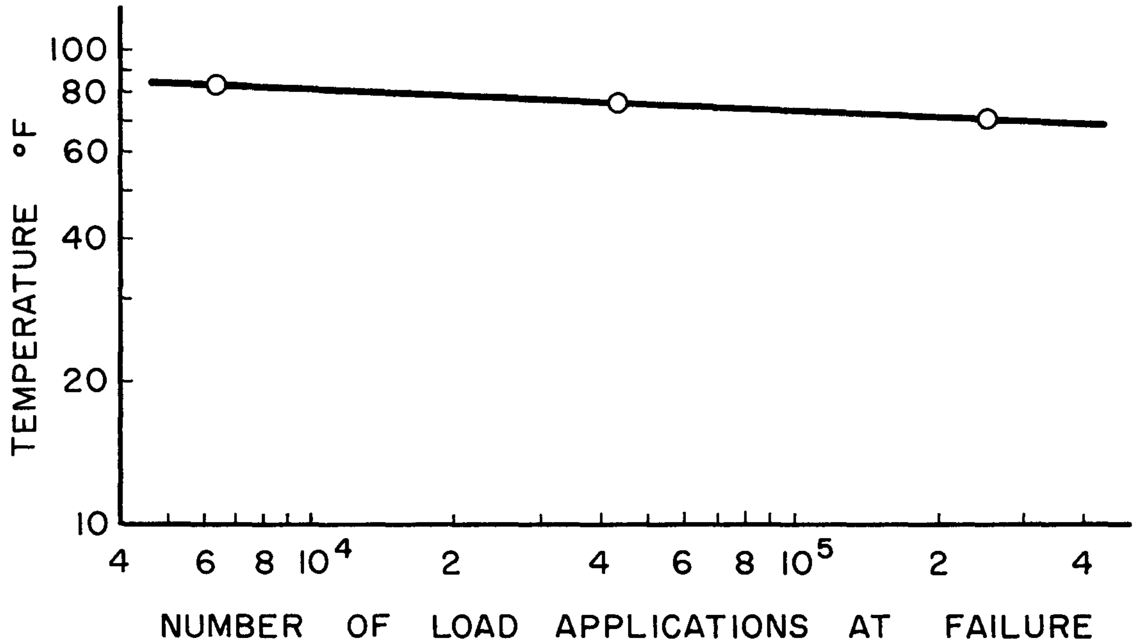
Figure 20

equivalent temperatures found for the viscosities of Figure 20 resulted in the curve of Figure 21. The relationship between temperature and number of load applications to cause failure implied by the curve of Figure 21 has not been verified in this research, but the trend does seem logical.

There is no evidence of which the writer is aware that differences in the penetration grade of paving asphalts affect the flexibility of asphaltic pavement surfacings. It is possible that visible surfacing failures are due primarily to factors such as strength of the road foundation or the design of the surfacing and that these mask any contribution to failure resulting from the original penetration value of the asphalt in the surfacing. Further, it is not believed that good or poor flexibility of an asphaltic concrete can be determined only from the original penetration value of the asphalt. Of more importance is the rate of hardening of the asphalt in service.

Aggregate Surface Texture

A comparison on number of load applications at failure was obtained for mixtures with coarse-textured aggregate (R-1) and also with smooth-textured aggregate (R-4). The curves of Figure 14 show clearly the greater resistance to the applied loads of the R-1 mixture. The differences in strengths appear comparable to the differences in the cohesiometer values for these mixtures (Appendix C, Table 9).



EQUIVALENT TEMPERATURE VS. NUMBER OF LOAD APPLICATIONS AT FAILURE

R-1-7.5-STANDARD

Figure 21

Specimen Density

Saal and Pell (23) showed that the fatigue life of a particular asphaltic concrete mixture decreased as the void content increased. In the present study the effect of specimen density on the ability of specimens to withstand the applied loads was not obvious, since for a void range of 4.4 to 13.0 percent, the number of load applications at failure varied from 40,000 to 45,000 (R-1-7.5-90). This is not to say that these findings are in disagreement with the statement of Saal and Pell. The specimens of Saal and Pell were of constant dimensions while the specimens in this phase of the present study were of constant weight. Using a constant weight of mixture for the specimens and various compactive efforts resulted in specimens of different densities and also thicknesses; that is, the specimens of higher densities had lower heights. It appears that any loss in strength due to increase in void content was offset by an increase in section modulus of the specimen.

Deflection of Test Specimens

The deflections discussed in the following paragraphs will be primarily the deflections of the load-disc resulting from the testing of specimens. The specimen deflections resulting from load are thought to have two components, one due to flow or yielding which is or should be time dependent and the other due to the elastic properties of the compacted mixture. The curves of Figure 13 show these two deformations. The influence of the support pressure on the amount of the repeated deflection is not known since the support pressure increased with number of load applications as did the magnitude of the repeated deflection. It is possible that the increase in repeated deflection as the test progressed was brought about by dilation of that portion of the specimen directly under the load disc. Had this been the primary reason, the volumetric expansion must have been very minor since density determinations performed after testing did not indicate any increase in volume of the specimen (Appendix C, Table 10). It is probable that the increase of repeated deflection was affected by the development and growth of slip planes. In the following paragraphs will be discussed deflections in the order of the data presented in Appendix C, Table 11.

R-1-90 asphalt content. The total accumulated deformations at failure of specimens tested at the five different asphalt contents were practically identical in that the range was from 0.195 inch for the mixtures with 6.0 percent asphalt to 0.215 inch for the specimens with 8.0 percent asphalt. A trend is indicated in using the accumulated deflections obtained after the first minute and one-half of loading (1000 repetitions). This amount of deflection increased as the asphalt content of the specimens increased, indicating that the mixtures with the greater percentage of asphalt were more flexible.

The repeated deflections of the load-disc at failure for these specimens were approximately 0.020 inch.

R-1-7.5-90 specimen thickness. The data show that the total accumulative deflection was greatest (0.240 inch) for the thinnest specimens and decreased (to 0.135 inch) as the thickness increased.

The repeated deflection at failure for the thinner specimens (1.0 inch) was approximately 0.040 inch and for the thicker one (1.45 inch) about 0.010 inch.

R-1-7.5-90 load disc. Statements have been made in a previous paragraph to the effect that no simple relationships were established for the variable of load contact data.

R-1-7.5 asphalt grade. Asphalt consistency showed an effect on deflections in that the softer the asphalt the greater the total accumulated deflection at failure of the specimen. These amounts were 0.240, 0.200 and 0.180 inch for specimens with asphalt contents of 7.5 percent and penetration grade 120-150, 85-100, and 60-70, respectively. There was no appreciable difference in the amounts of accumulated deflections added from 1000 load applications to failure for the specimens made with the three different grades of asphalt; neither was there much difference in the amount of repeated deflection at failure.

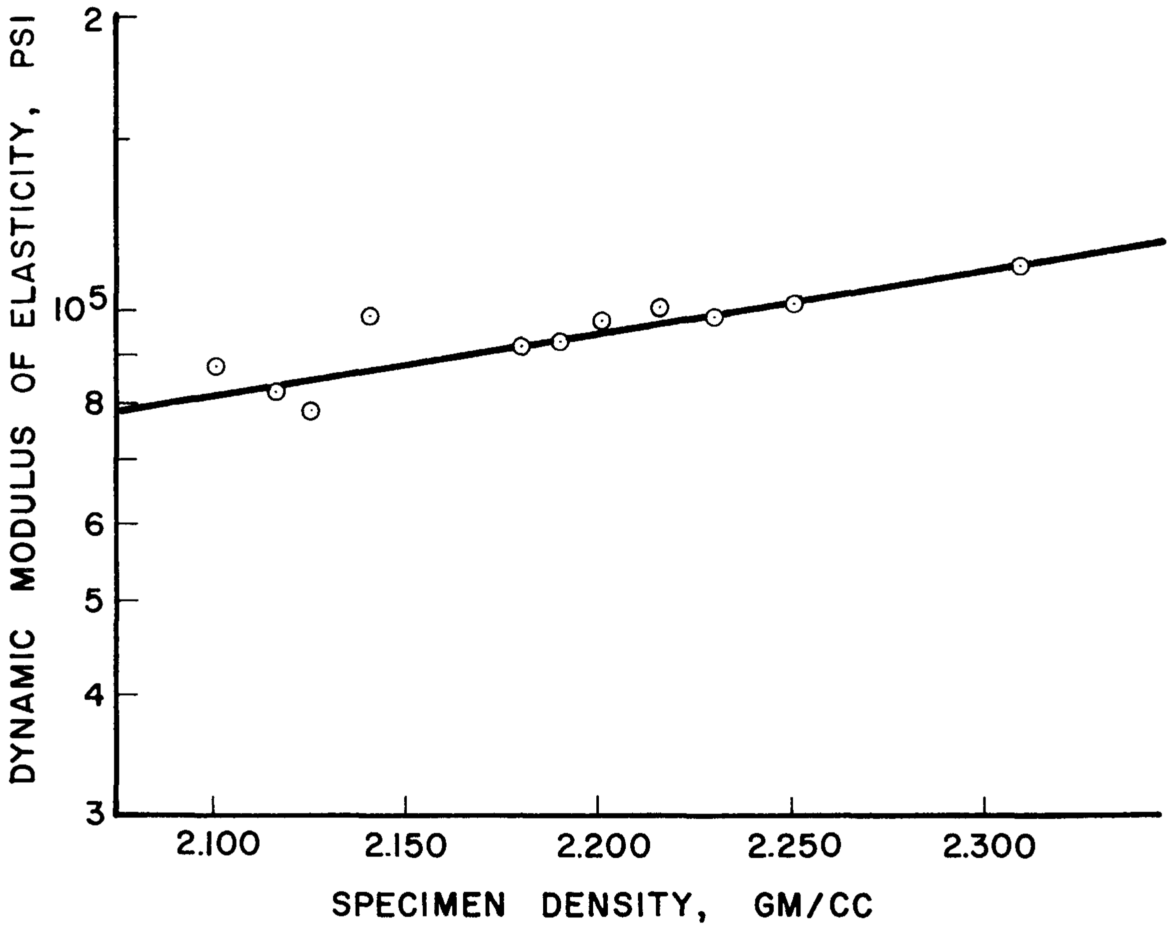
Use of Grashof's Equations

In a previous section the flexural formulae of Grashof were cited, and their use and simplification for utilization in this program were illustrated. The computed values of stress, strain, and modulus of elasticity will be presented in the next paragraphs.

Dynamic modulus of elasticity. It has been stated that the dynamic moduli of elasticity were computed for the condition when the specimen support was at 1.5 pounds per square inch and the deflection employed for the computation was the repeated load-disc deflection occurring at the above support pressure (equation 4). The amount of load-disc repeated deflection varied with the thickness of specimen and ranged from 0.008 to 0.027 inch.

The curve of Figure 22 shows a trend for this increase of dynamic modulus of elasticity as the specimen density increases. Limited data also showed that the modulus increased as the viscosity of the asphalt in the mixture increased. These relationships have previously been determined by others as has been stated in the literature survey (2). The general average value of 100,000 pounds per square inch at 75°F shown for the R-1-7.5-90 mixture is higher than values of moduli of elasticity found by other investigators (10,21), these being between 30,000 and 50,000 pounds per square inch; however, the WASHO report (15) showed a dynamic modulus of elasticity of approximately 1,500,000 pounds per square inch at 75°F. Differences in mixtures and rates of loading may account for these variations.

Stresses and strains. Radial stresses and strains were evaluated at the center and surface of a specimen for a load condition identical to the one used for computation of the dynamic modulus of elasticity. The plots of Figures 23 and 24 show the relationships found between central radial stress and strain versus number of load applications at failure. The linear relationship between stress and number of load repetitions to produce failure on log-log coordinates has previously been presented by Papazian and Baker (21). The work of Saal and Pell (23) showed a trend similar to that presented in Figure 24 in that the greater the applied strain the shorter the fatigue life of a specimen.

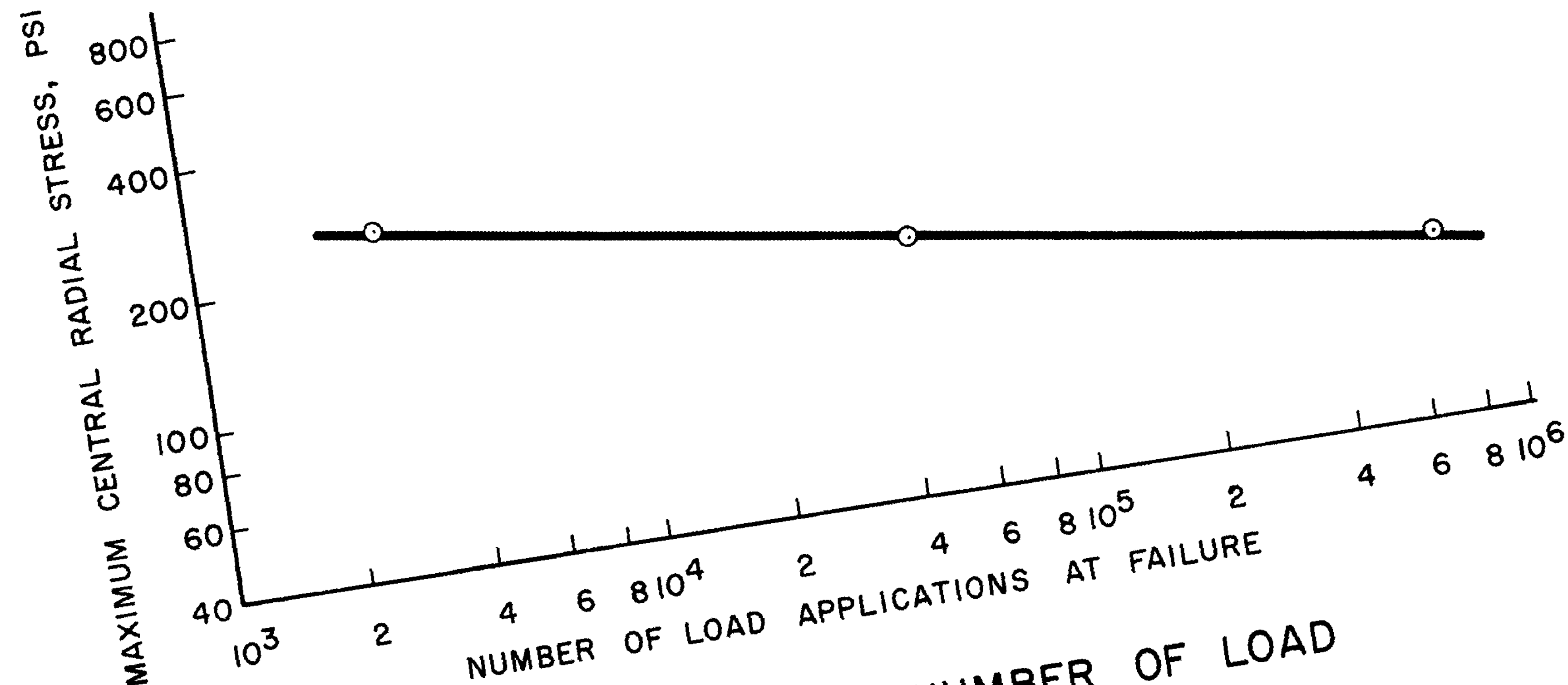


DYNAMIC MODULUS OF ELASTICITY

VS. SPECIMEN DENSITY

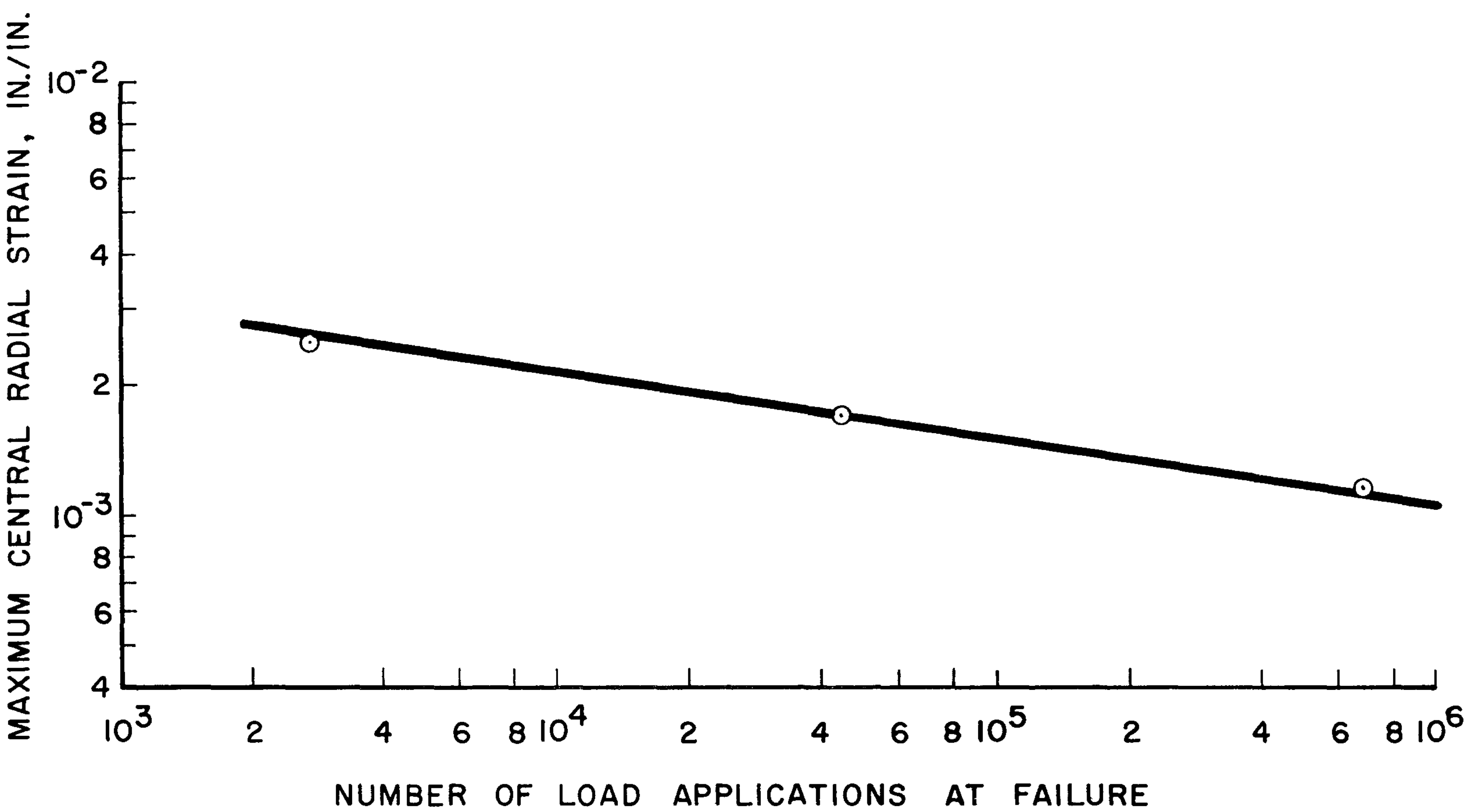
R-1-7.5-90

Figure 22



MAXIMUM STRESS VS. NUMBER OF LOAD APPLICATIONS AT FAILURE

Figure 23



MAXIMUM STRAIN VS. NUMBER OF LOAD APPLICATIONS AT FAILURE

Figure 24

The curves of Figures 23 and 24 are approximately parallel to each other; thus it seems that either stress or strain might be used to establish endurance limit of asphaltic concrete.

SUMMARY AND CONCLUSIONS

The following comments will serve to summarize and recapitulate the achievements of this investigation.

The Deflectometer as a Compaction Apparatus

The compaction procedure established for molding test specimens employing the loading system of the deflectometer was adequate for obtaining the desired densities and aggregate arrangement. The most significant factor in densifying the asphaltic mixtures appears to be the application of horizontal forces during the dynamic loading period. The compaction method produced specimens that were uniform in density throughout the compacted mixture; the large 17 1/2-inch and 4-inch diameter specimens produced were of comparable densities and particle orientation for any one mixture. However, the loading system of the deflectometer should be of heavier construction for the purpose of compacting the large specimens.

The Deflectometer as a Testing Apparatus

Evaluation of the deflectometer for testing large asphaltic concrete specimens subjected to repeated flexures has shown that the apparatus yields reproducible results and that it is sensitive to factors affecting the flexibility of asphaltic concrete. Despite the acceptable performance of the deflectometer, it should be recognized that it is a laboratory apparatus for the evaluation and comparison of different asphaltic mixtures under controlled conditions. A tentative standard method of testing was established for this research which may need modification for correlation of laboratory results with service performance of asphaltic concrete pavement surfacings.

Factors Affecting the Flexibility of Asphaltic Concrete

The study has indicated basic relationships between specimen variables and resistance to repeated flexures of molded samples. The trends established are as follows.

1. The resistance of a mixture to repeated loads increases as the asphalt content increases to an optimum amount of asphalt, and then the resistance diminishes with further increase of asphalt.
2. The thickness of specimen influences the endurance to repeated flexures by affecting the amount of induced strain or stress for a constant load.
3. The stress induced on a specimen by a fixed load is dependent on the amount of support given to the specimen.
4. The use of an asphalt of lower consistency may decrease the endurance of a mixture to repeated flexures but results in a more flexible mixture.
5. The resistance of asphaltic mixtures to repeated loads can be maximized by the use of (a) coarse-textured aggregates, (b) high asphalt content (within limits), and (c) by the use of high viscosity asphalt.
6. The flexure formulae of Grashof are suitable for the evaluation and comparison of stresses corresponding to the endurance of different asphaltic concrete mixtures with respect to repeated flexures.

SUGGESTIONS FOR FURTHER RESEARCH

Results of the work performed for this research have been for fine-grained asphaltic concrete mixtures. It would be desirable to determine if the general relationships established apply to coarse-grained mixtures and for different gradations of aggregate.

Since a primary objective of the research was to evaluate the deflectometer and since the data obtained are limited in amount, specific variables such as specimen thickness or induced strain or stress should be given greater coverage to determine if a definite endurance limit exists for asphaltic concrete mixtures.

Of great importance and need is the correlation of laboratory data obtained utilizing the deflectometer with service performance of asphaltic concrete surfacings. Since the endurance of asphaltic concrete is largely dependent on the support given by the base material, it appears desirable to establish procedures for determining the anticipated support given by a base and then design the asphaltic surfacing with respect to mixture composition and thickness to be placed.

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APPENDIX A

Calibration of Reaction Unit

CALIBRATION OF REACTION UNIT

The purpose of calibrating the reaction unit is to insure that a constant amount of air is in the system. This calibration is necessary so that specimens bending comparable amounts during the testing operation receive equal amounts of support from the oil in the reaction unit.

A light weight motor oil (SAE-10) was used in the reaction system, but most lubricating oils will serve the same purpose. The oil is introduced into the chamber through a tee connection in the line joining the pressure gage and the reaction unit. Approximately 7.5 gallons of oil are required. During filling, the unit is tilted so that the bleeder valve secured to the chamber is at the high elevation and remains open. The rubber diaphragm is manipulated to force air out through the bleeder valve. After the oil chamber and pump have been filled, the bleeder valve is closed and the reaction chamber is positioned on the pedestal. Subsequent addition of oil or air to the system is introduced through the pressure gage tee. Removal of air in the chamber is achieved in the manner described with use of the bleeder valve. Air in the pump is removed by first closing the angle valve connecting to the chamber and then opening the valve on the tee which unites the pump hose and angle valve.

The system is checked for the proper amount of air in accordance with the following procedure:

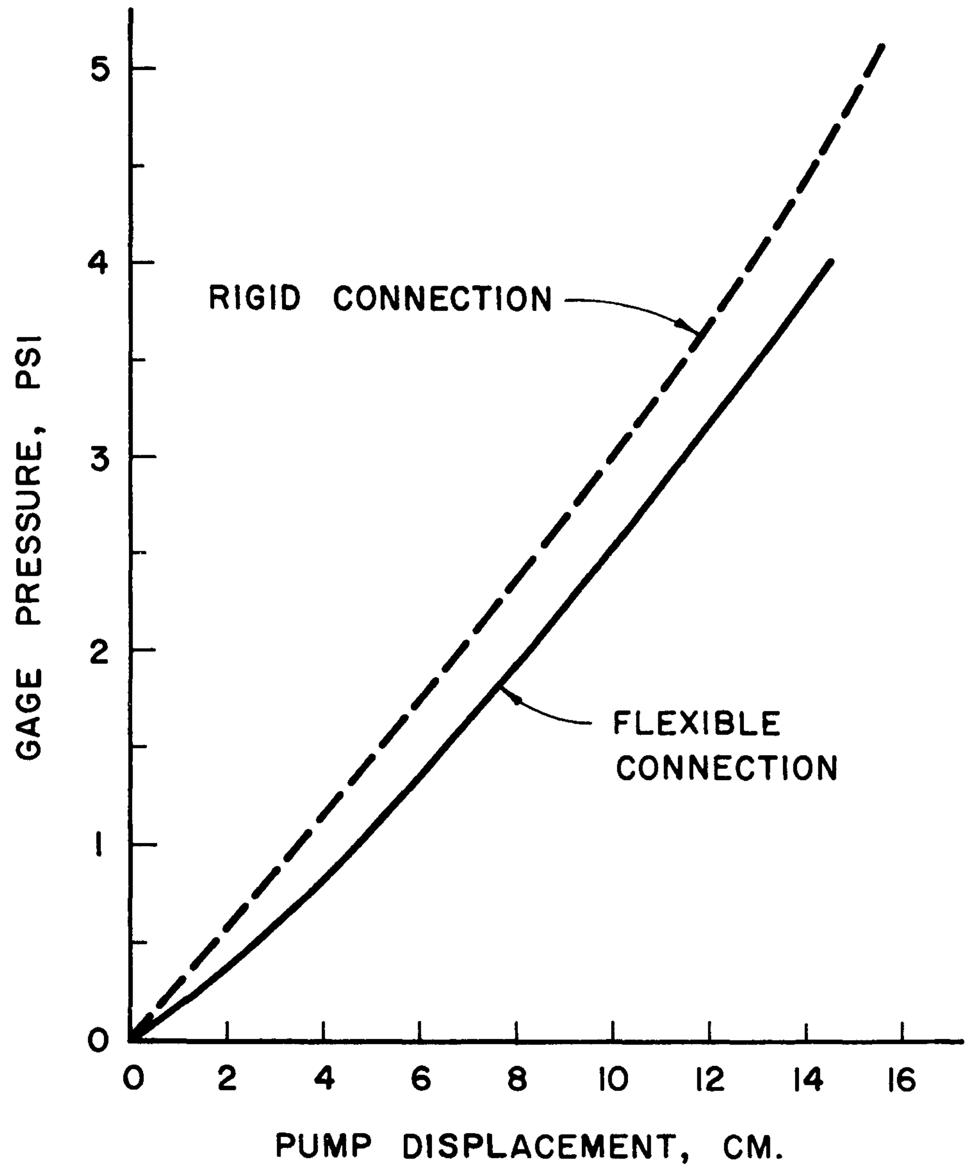
1. Allow the system to reach equilibrium at the desired test temperature.

2. Secure the 3/4-inch thick by 17 1/2-inch diameter steel plate onto the reaction unit. Use all bolts and draw these to the tension controlled by the spacer sleeves and springs.

3. Open the angle valve and adjust the position of the pump piston so that the pressure gage registers 1.0 pound per square inch and note the reading indicated on the scale of the pump.

4. Displace the pump piston 5 centimeters to cause an increase of pressure in the oil chamber. This displacement of oil volume, corresponding to 5.08 cubic inches, should raise the oil pressure reading to 2.50 pounds per square inch, if the proper amount of air is in the system. Another equal increment of piston movement will then result in a gage reading of 4.05 pounds per square inch. A tolerance of 0.1 pound per square inch is allowed.

The above changes in pressure created by the displacements of the pump piston are based on results obtained when the pump was connected to the oil chamber with a flexible air hose (1/2 inch). Other values will be obtained when this connection is effected with rigid pipes. Calibration curves are shown for the two conditions in Figure 25.



CALIBRATION OF DEFLECTOMETER

Figure 25

APPENDIX B

Vibratory-Kneading Compaction Procedures

COMPACTION PROCEDURES

Prior to starting compaction of a specimen, the deflectometer requires certain modifications in order to perform as a compactor. The following additions are necessary:

1. Place the 2-inch wooden spacer on the reaction unit and secure with the steel ring and bolts.
2. Add ballast plates totaling 91 pounds and screw on the built-up 17 1/2 inches in diameter compaction head.
3. Omit proving ring and revolution counter.
4. The weights of the eccentrics are C, 3, 4, 5, and 6 both right and left.
5. Select the combination of pulleys on the motor and arbor resulting in a loader speed of 1200 revolutions per minute.
6. Attach the work table to the reaction unit.

The deflectometer is now ready for molding the large 17 1/2-inch diameter specimens. The mixture to be molded is brought to the molding temperature of 300°F in sealed containers placed in a forced-draft oven. The split-ring mold, bottom 17 1/2-inch diameter plate, and the compaction head are heated before the start of compaction. Placing of the mix in the mold and compaction proceed in the following manner:

1. Set a transfer board on the work table with the steel disc and mold. Place a pallet (1/4-inch thick by 17 1/2-inches in diameter fiber board) and then a thin (.010 inch) metal disc or embossed aluminum foil on the steel plate.

2. Spread a portion of the hot asphaltic mixture about 1 inch thick into the mold and with chopping action use the edge of a warm metal trowel to force the fines to the bottom.
3. Spread the remainder of the mixture in a manner similar to item 2 except that penetration of the trowel is limited to the upper layer. Smooth and level the top surface.
4. Compact with a 6-inch diameter tamper, lightly and once over the surface, making certain that there is no mix above the surface around the edges. The gradation of certain mixtures requires that a grid tamper be used to bring the fines to the top surface in order to obtain a smooth face.
5. Place a paper disc on the top surface and slide the mold onto reaction unit.
6. Center the mold and lower the loading assembly into the mold; then secure the mold to the reaction unit by means of turnbuckles hooking to the handles of the mold and bolt-holes on the shoulder of the oil chamber.
7. Relieve the tension on the elevating cable and start the loading motor.
8. Oscillate the loading head through an angle of about 40 degrees during the compaction period. The vibratory compaction period depends upon the amount of mix and is computed on the basis of 3 minutes per 10,000 grams, but not to exceed 4 minutes.
9. Raise and secure the loading head after vibrating, release the turnbuckles, and slide the mold onto the transfer board.

10. Center the mold on a testing machine with a rigid loading head 17 1/2 inches in diameter and capable of applying 250,000 pounds. The leveling load is slowly (in about 1 minute) increased to 234,000 pounds (1000 pounds per square inch) and held for 2 minutes and then released.
11. Remove the mold from the testing machine and rest it on supports so that the mold may be pushed free of the specimen and steel disc.
12. Slide the specimen off the steel plate by pushing on the fiber pallet. The upper paper disc is removed, the specimen is marked on the upper face for identification, and allowed to cool to room temperature before transferring to storage.
13. Remove the bottom sheet metal or foil after the specimen has cooled. Slight heating may be necessary to remove the foil.

Compaction of the 4-inch diameter specimens is accomplished in a similar manner to that described above. The specimens are compacted in Proctor molds as follows:

1. Use the 4-inch diameter head on the loading unit and a loading speed of 960 revolutions per minute.
2. Bolt two clamping bars onto the reaction unit to secure the base plate of the Proctor mold
3. Introduce the hot mix into the Proctor mold in which a 4-inch paper disc has previously been placed. The mix is spaded around its sides with a spatula and leveled with a large bent spoon. Another paper disc is placed on the upper surface.

4. Center and secure the mold on the reaction unit. The loading system is released and vibrated with continuous oscillations of the head for two minutes. The height of specimens desired is either 2 or 2 1/2 inches, but the period of vibration for producing these is the same.
5. Transfer the mold to the jack assembly (or testing machine) after removal from the deflectometer. Apply load to the upper surface of the specimen. The load is 12,500 pounds (1000 pounds per square inch) and is indicated by the jack gage at a pressure of 1560 pounds per square inch.
6. Extrude the specimen from the mold, remove the paper discs, and mark the upper face for identification.

APPENDIX C

Summary of Data for Stability Tests and Repetitive Loading Tests Specimens

Table 8

Design Values of Mixtures Molded by the Texas Gyratory-Shear Method

Asphalt Content %	Specimen Density gm/cc	Relative Density %	Hveem Stability %	Cohesiometer Value gm/in. width	Vol. Asp. = Agg. Area F.T. micron
		R-1-135			
7.1	2.259	92.8	40	190	
7.5	2.297	95	41	241	
		R-1-90			
6.0	2.341	95.3	35	313	6.8
6.5	2.363	96.8	36	321	7.4
7.1	2.371	97.4	29	413	8.1
7.5	2.357	97.6	26	448	8.6
8.0	2.352	97.8	16	400	9.2
		R-1-65			
7.1	2.273	93.5	41	339	
7.5	2.258	93.5	40	364	
		R-4-90			
6.0	2.225	92.3	30	49	
6.5	2.244	94.5	32	60	
7.1	2.253	95.6	26	70	

Table 9

Design Value of Mixtures Molded by the Vibratory-Kneading Method

Asphalt Content %	Specimen Density gm/cc	Relative Density %	Hveem Stability %	Cohesiometer Value gm/in. width
		R-1-135		
7.1	2.151	88.5	31	152
7.5	2.160	89.5	30	152
		R-1-90		
6.0	2.228	90.6	35	407
6.5	2.225	91.5	32	258
7.1	2.261	93.4	32	306
7.5	2.291	95	31	339
8.0	2.340	97.4	29	537
		R-1-65		
7.1	2.142	88	28	292
7.5	2.154	89.4	27	266
		R-4-90		
6.0	2.129	88.5	28	60
6.5	2.150	90.6	26	40
7.1	2.161	92.0	24	

Table 10

Repetitive Load Test Specimen Data

Asphalt Content % A		Thick. in. B	Before Test gm/cc C	After Test gm/cc D	Impregnated Sp.Gr. E	Total Voids Before Test % F	Ratio, Reps Applied / Reps to Fail G	
			Variable	R-1-90 Asphalt Content				
6.0	A	1.19	2.221	2.214	2.455	13.4	6.2	
	B	1.20	2.188	2.177	"	14.8	3.3	punctured
	C	1.21	2.180	2.173	"	15.2	5.7	
6.5	A	1.22	2.215	2.211	2.440	9.4	5.4	
	B	1.25	2.209	2.202	"	9.5	3.2	
	C	1.22	2.216	2.215	"	9.4	3.6	
7.1	A	1.20	2.231	2.232	2.435	8.4	2.9	
	B	1.21	2.228	2.231	"	8.5	2.9	
	C	1.20	2.232	2.226	"	8.4	4.1	
7.5	A	1.20	2.251	2.249	2.416	6.9	2.6	
	B*	1.18	2.278	2.278	"	5.6	2.8	
	C	1.17	2.310	2.307	"	4.4	2.1	
8.0	A	1.20	2.297	2.285	2.405	4.5	3.2	
	B	1.17	2.325	2.312	"	3.4	2.8	
	C	1.19	2.298	2.292	"	4.4	4.0	

*Test temperature of 80°F

Asphalt Content % A		Thick. in. B	Before Test gm/cc C	After Test gm/cc D	Impregnated Sp.Gr. E	Total Voids Before Test % F	Ratio, Reps Applied / Reps to Fail G
			Variable	R-1-90 Specimen Thickness			
7.5	A	0.96	2.231	2.232	2.416	7.6	2.1
	B	0.97	2.215	2.210	"	8.4	2.5
	C	0.97	2.201	2.191	"	8.8	1.5
7.5	A	1.42	2.191	2.185	"	9.3	1.7
	B*	1.46	2.166	2.162	"	10.4	1.3
	C	1.45	2.178	2.177	"	9.9	1.6

*Test temperature of 80°F

continued ...

Table 10 (cont'd)

A	B	C	D	E	F	G
		Variable	R-1-90 Load Disc			
7.5 A	1.25	2.166	2.186	2.416	10.4	1.8
" B	1.22	2.217	2.216	"	8.4	2.0
7.5 A	1.26	2.147	2.144	"	11.1	1.6
5.0 C	1.26	2.140	2.145	"	11.4	2.5
7.5 B	1.24	2.174	2.159	"	10.0	3.9
8.0 C	1.25	2.163	2.164	"	10.4	2.3
		Variable	R-1-90 Initial Support			
7.5 A	1.27	2.116	2.132	2.416	12.4	1.1
0.0 C	1.27	2.125	2.127	"	12.0	1.8
7.5 B	1.29	2.100	2.109	"	13.0	1.3
1.0 C	1.27	2.112	2.123	"	12.6	1.4
7.5 A	1.25	2.129	2.142	"	11.9	3.1
2.0 B	1.24	2.146	2.155	"	11.1	2.2
		Variable	R-1-65 Asphalt Content			
7.1 A	1.25	2.139	2.147	2.438	12.3	2.0
B	1.24	2.139	2.141	"	12.3	1.5
C	1.23	2.156	2.156	"	11.6	2.2
7.5 A	1.25	2.156	2.155	2.413	10.6	1.7
B	1.26	2.130	2.131	"	11.7	1.6
C	1.27	2.117	2.106	"	12.3	2.0 punctured
		Variable	R-1-135 Asphalt Content			
7.1 A	1.22	2.135	2.137	2.433	12.2	1.4
B	1.21	2.162	2.158	"	11.0	2.0
C	1.23	2.106	2.106	"	13.2	1.4
7.5 A	1.23	2.181	2.184	2.419	9.6	1.9
B	1.24	2.164	2.166	"	10.4	3.8
C	1.24	2.180	2.179	"	9.6	2.7

continued ...

Table 10 (cont'd)

A	B	C	D	E	F	G
		Variable	R-4-90 Asphalt Content			
6.0 A	1.22	2.118	2.121	2.410	12.2	1.5
B	1.21	2.133	2.126	"	11.6	1.5
C	1.25	2.112	2.106	"	12.5	1.5
6.5 A	1.21	2.148	2.152	2.375	7.8	1.4
B	1.23	2.128	2.133	"	10.2	1.7
C	1.22	2.133	2.136	"	10.1	1.7
7.1 A	1.23	2.146	2.146	2.356	8.9	3.0
B	1.19	2.188	2.188	"	7.1	2.4
C	1.21	2.154	2.154	"	8.5	2.7

Table 11

Summary of Repetitive Load Test Data
(D.L. = 139#, F_r = 104#, Frequency = 660 RPM, Temperature = 75°F)

Specimen		Loading		Support Pressure at 1.5 psi		Loading Disc Deflections and Repetitions				
Asphalt Content %	Thick. in.	Disc Area sq.in.	Initial Support psi	Load Reps	Deflect. Recov. in.	Deflct. at 1000 reps, D_{1000}, in.	Deflct. at Failure, D_F in.	D_F-D_{1000} in.	Reps to Failure	Deflct. Recov. at Fail. in.
A	B	C	D	E	F	G	H	I	J	K
				Variable	R-1-90 Asphalt Content					
6.0 A	1.19	5.00	1.00	1600	.016	.116	.172	.056	6700	.022
B	1.20	"	"	400	.014	.165	.240	.075	6000	.030
C	1.21	"	"	800	.016	.116	.172	.056	6700	.022
6.5 A	1.22	"	"	1200	.013	.113	.182	.070	11,000	.018
B	1.25	"	"	700	.013	.142	.236	.093	13,000	.020
C	1.22	"	"	1000	.011	.130	.208	.078	11,500	.018
7.1 A	1.20	"	"	1500	.012	.130	.240	.110	23,000	.022
B	1.21	"	"	2400	.013	.102	.210	.108	21,000	.024
C	1.20	"	"	2000	.011	.087	.158	.071	22,000	.016
7.5 A	1.20	"	"	2200	.012	.096	.210	.114	45,000	.019
B*	1.18	"	"	1000	.016	.140	.260	.120	13,000	.024
C	1.17	"	"	2000	.011	.084	.185	.101	43,000	.016
8.0 A	1.20	"	"	1000	.011	.110	.220	.110	28,000	.020
B	1.17	"	"	1500	.012	.120	.242	.121	25,000	.022
C	1.19	"	"	1500	.011	.092	.182	.090	27,500	.019

*Test Temperature of 80°F.

continued ...

Table 11 (cont'd)

A	B	C	D	E	F	G	H	I	J	K
Variable R-1-90 Specimen Thickness										
7.5 A	0.96	5.00	1.00	600	.027	.179	.240	.061	2,800	.041
B	0.97	"	"	1,000	.026	.156	.195	.039	2,800	.032
C	0.97	"	"	200	.028	.210	.274	.064	2,600	.054
7.5 A	1.42	"	"	20,000	.008	.054	.152	.098	560,000	.010
B*	1.46	"	"	10,000	.009	.054	.224	.170	320,000	.014
C	1.45	"	"	350,000	.008	.035	.120	.085	800,000	.009

*Test temperature of 80°F

A	B	C	D	E	F	G	H	I	J	K
Variable R-1-90 Load Disc										
7.5 A	1.25	3.14	1.00	2,500	.010	.091	.228	.137	33,000	.022
B	1.22	"	"	4,000	.013	.101	.206	.105	32,000	.020
7.5 A	1.26	5.00	"	3,000	.016	.115	.240	.125	40,000	.026
C	1.26	"	"	3,000	.011	.132	.220	.088	45,000	.016
7.5 B	1.24	8.00	"	4,500	.010	.073	.140	.067	130,000	.014
C	1.25	"	"	7,000	.012	.095	.185	.090	145,000	.018
Variable R-1-90 Initial Support										
7.5 A	1.27	5.00	0.00	20,000	.013	.150	.275	.125	28,000	.021
C	1.27	"	"	25,000	.014	.150	.275	.125	28,000	.021
7.5 B	1.29	"	1.00	4,000	.012	.083	.170	.087	45,000	.018
C	1.27	"	"	3,000	.013	.107	.200	.093	42,000	.019
7.5 A	1.25	"	2.00	-	-	.053	.108	.055	58,000	.015
B	1.24	"	"	-	-	.059	.111	.052	54,000	.016

continued ...

Table 11 (cont'd)

A		B	C	D	E	F	G	H	I	J	K
					Variable	R-1-65 Asphalt Content					
7.1	A	1.25	5.00	1.00	6,000	.008	.064	.175	.111	230,000	.014
	B	1.24	"	"	6,000	.010	.076	.200	.124	200,000	.018
	C	1.23	"	"	2,000	.009	.080	.215	.135	210,000	.020
7.5	A	1.25	"	"	38,000	.010	.065	.182	.117	280,000	.014
	B	1.26	"	"	25,000	.011	.065	.182	.117	280,000	.018
	C	1.27	"	"	4,000	.011	.088	.260	.172	200,000	.022
					Variable	R-1-135 Asphalt Content					
7.1	A	1.22	5.00	1.00	500	.013	.170	.255	.085	10,000	.022
	B	1.21	"	"	1,000	.014	.135	.220	.085	15,000	.021
	C	1.23	"	"	500	.016	.195	.285	.090	10,000	.025
7.5	A	1.23	"	"	400	.018	.210	.290	.080	6,800	.024
	B	1.24	"	"	1,000	.014	.140	.190	.050	6,500	.016
	C	1.24	"	"	600	.016	.185	.238	.053	6,000	.019
					Variable	R-4-90 Asphalt Content					
6.0	A	1.22	5.00	1.00	800	.014	.135	.225	.090	8,300	.020
	B	1.21	"	"	800	.015	.150	.245	.095	8,200	.021
	C	1.25	"	"	800	.014	.135	.215	.080	8,000	.020
6.5	A	1.21	"	"	500	.016	.170	.251	.081	6,500	.020
	B	1.23	"	"	500	.016	.170	.260	.090	6,000	.022
	C	1.22	"	"	500	.016	.170	.245	.075	7,000	.020
7.1	A	1.23	"	"	3,000	.013	.094	.165	.071	15,000	.017
	B	1.19	"	"	200	.020	.230	.280	.050	2,500	.025
	C	1.21	"	"	400	.018	.172	.210	.038	3,000	.021