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SECOND ANNUAL REPORT

THE MOISTURE MECHANISM THAT CAUSES ASPHALT STRIPPING IN ASPHALTIC PAVEMENT MIXTURES

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and
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THE MOISTURE MECHANISM THAT CAUSES ASPHALT STRIPPING IN ASPHALTIC PAVEMENT MIXTURES

by

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Efforts of students at the University of Idaho were a significant factor in making test specimens, in conducting tests and in reporting test data. These students are:

Dennis Johnson, Civil Engineering -- Weiser, Idaho
(MSCE, September 1969 -- Thesis: "Debonding of Water-Saturated Asphaltic Concrete Caused by Thermally Induced Pore Pressure.") (Research Fellow).

Henry Fan, Civil Engineering -- Hong Kong
(MSCE, September 1969).

Roger Chen, Civil Engineering -- Taiwan

Kerby Cole, Civil Engineering -- Moscow, Idaho

Mike Hagler, Civil Engineering -- Meridian, Idaho

Roger Hopt, Civil Engineering -- Boise, Idaho

Ken Thomas, Civil Engineering -- Boise, Idaho

Carl Ellsworth, Civil Engineering -- Boise, Idaho

I. INTRODUCTION

The interaction of water and asphaltic concrete may under particular circumstances cause stripping or loss of adhesion and consequential detachment of the asphalt from the aggregate. The result of this action decreases the cohesive strength of the mixture until it has no inherent structural strength as a paving material and it approaches the condition of compacted gravel.

Since asphaltic concrete is a nonhomogeneous material, many factors can contribute to the overall stripping problem. Built-in porosity and permeability allow water to enter the mixture and flow through the void paths. The type of asphalt cement and the surface characteristics of the aggregates are responsible for adhesive strength; the adhesive strength may not be adequate in the presence of internal water.

Although much research is being done on the stripping problem, few investigations have been made on compacted asphaltic mixtures. It was found in the Phase I experimentation that the use of compacted mixture specimens in the testing program is a more direct approach than use of loose asphalt-aggregate mixtures because it more closely simulates the conditions of field use. Results with compacted mixtures therefore appear to be more informative to the engineer desiring to predict stripping problems in pavements. This is because internal void water or pore pressures, for example, can be created realistically in compacted mixture specimens, producing forces which tend to strip the mixtures. It is hypothesized that these pressures, similar to those in actual pavements, cause the stripping when the asphalt-aggregate interface is weak.

The objective of Phase II of the current research project is to formulate the stripping mechanism as occurring in Idaho pavements using test results obtained in the laboratory and observations made in the field.

Two aggregate sources were used in the research to formulate the mechanism.

Inkom aggregate BK142-S (used in a paving project which later showed stripping failure) was used for almost all test specimens, and Washington No. 28 aggregate (potentially non-stripping) was used for the remaining test specimens. Four asphalts marketed in Idaho were used with the two aggregates.

Five control specimens of each asphalt-Inkom aggregate combination were freeze-thaw cycled and examined for amount of debonding at varying numbers of cycles. These specimens provided comparison for the variate specimens which were made later. Control variate specimens either contained chemical additives designed to increase adhesion strength, contained different gradations of Inkom aggregate, contained a different amount of asphalt, or contained Washington No. 28 aggregate. Debonding in the variates was compared to the debonding in the control specimens.

Asphalt-aggregate combinations similar to the control specimens were examined in the void pressure determinations. The approach involved measuring both volumetric strain and bulk tensile modulus of the asphaltic concrete. About 25 specimens were used in the volumetric strain determinations while about 50 smaller specimens were used in the modulus determinations.

The overall mechanism is reported and two possible analytical approaches are outlined.

II. LITERATURE SURVEY

A recent Idaho study, published in 1969, was made to determine the extent of stripping in Idaho (1)[1]. Pavement samples from throughout the State were taken and examined for stripping in both the +4 and -4 sieve size aggregate particles. The major conclusion made from the study was that no single mixture or location factor appeared to be responsible for the observed stripping. Quartzite, sandstone and limestone were the most frequent aggregates identified with stripping, and all of the penetration grade (high viscosity) asphalts except two samples of 200/300 penetration grade asphalt were identified with stripping. This study pointed out the widespread nature of the stripping problem, and the need for identifying the mechanism of stripping in the case that a nonapparent combination of two or more factors is responsible.

A special study group was formed on the West Coast to examine stripping in problem aggregates from Idaho and Arizona. Using conventional test procedures on loose aggregate, each of the cooperating agencies tested and reported on the stripping potential of these aggregates. Idaho's Inkom aggregate which did show stripping in a highway pavement is of interest here.

A summary of action taken so far by the group may be found in the May 16, 1969, Minutes of Meeting (2). Testing by Arizona, California, Oregon and Washington indicated that the Idaho aggregate would be suitable for use under present standards.

California research as reported in a memorandum indicates that the Inkom aggregate is not susceptible to stripping as determined by conventional test methods on both loose and compacted mixtures (3). Their Immersion-Compression tests do show some stripping, but this was attributed to the presence of 5 percent montmorillonite in the aggregate.

It is seen that several agencies, acting independently, have shown the

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Note 1: Numbers underlined in parentheses refer to the Bibliography.

Inkom aggregate not to be susceptible to stripping. Yet the pavement constructed with this aggregate has shown extensive stripping failure. While conventional tests are valuable in predicting adhesion failure, it is felt they do not fully simulate field conditions since internal water action could not exist in a realistic form in loose mixtures.

Freeze-thaw or temperature cycling as a method for measuring resistance to stripping has not yet been used for water-saturated asphaltic mixtures. Ward K. Parr of the Michigan Department of Highways some time ago mentioned the possibility of freeze-thaw breakup in asphaltic concrete. The destructive effects of freezing on portland cement concrete have been known for some time, and T. C. Powers in 1945 postulated the existence of destructive hydraulic pressures generated by an advancing ice wall (4). It was thought that the formation of ice on the outside of the concrete and subsequent expansion of the ice would drive unfrozen water into the permeable voids under high pressure, thus causing tensile failure of the cement mortar.

The possibility of air pore pressures in nonsaturated asphaltic concrete undergoing temperature rise was advanced by Jones (5). He found that thermal coefficients of expansion were, on the average, larger than corresponding coefficients of contraction. He concluded that air pressures in the voids were causing creep of the mixture during a given temperature rise. Assuming that no vacuum formed in the voids on the contraction part of the cycle, lack of "reverse" pressure would show smaller coefficients of contraction. Jones did not attempt mathematical calculation of this effect due to the complexity of the situation.

Lee and Nichols have mentioned the possible existence of hydraulic pore pressures in pavement surfaces caused by the "pumping" action of moving vehicle wheels (6). They surmise that water will creep between the asphalt binder and the aggregate under this pumping action and, as a result, will cause debonding or stripping. Their approach was essentially centered around surface failure of pavements in the presence of water.

Measurement of thermally induced hydraulic pore pressures in soils has been performed by Plum and Esrig (7). They tested undrained clays in a triaxial cell with a lateral confining pressure of 30 psi at $57^{\circ}F$. By raising the temperature of the cell to $95^{\circ}F$, they noted a corresponding pore pressure increase to 45 psi. Additional cycles of thermal rise and fall between these two temperatures resulted in further increases of pore pressure to a maximum of 48 psi and eventually a closed hysteresis loop. This work serves as an example of existence and measurement of internal pore pressure induced thermally in a porous material and may be analogous to a saturated asphaltic concrete.

Nondestructive testing on compacted asphaltic mixtures was employed by Andersland (8). Here the sonic modulus of a beam of asphaltic concrete was measured after immersion in water for one, three, five and nine days. With this method it was not necessary to rely on visual estimation of the stripping since the sonic modulus decreases in proportion to the amount of stripping, and progressive failure may be observed on the same specimen. Specimens for this test must have a length to width ratio of three, and they are more difficult to prepare than the smaller conventional specimens.

Research and testing to date on stripping have centered on thermodynamic theoretical aspects of adhesion failure and on surface energy measurements and tests related to those concepts. A series of conventional tests on Inkom aggregate failed to reveal the full potential stripping nature of this aggregate. It appears that compacted mixtures are used in both destructive and nondestructive tests, but not to the advantage they could be. Reliance on visual estimation of stripping is eliminated in the sonic modulus test, and it may be a good test to use if the larger specimens can be conveniently prepared. Freeze-thaw cycling, although used in portland cement concrete testing, has not been used as a test method for predicting stripping in asphaltic concrete.

Based on the literature investigated and reviewed, it was believed that one approach to measure hydraulic void (pore) pressures in compacted, water-saturated asphaltic mixtures and to define this pressure relationship to stripping would be promising.

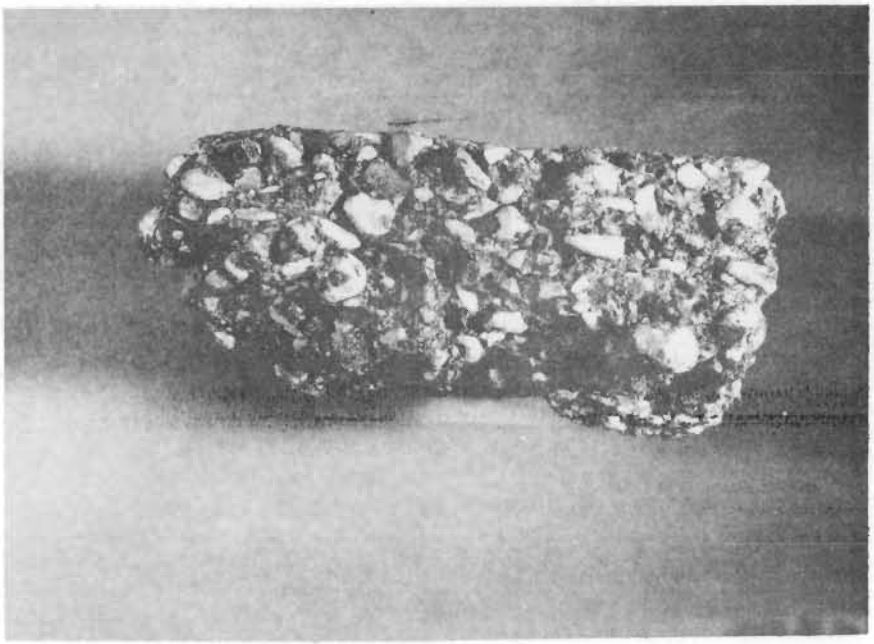
III. TEST OBSERVATIONS

This section is essentially divided into two parts. The first part concerns stripping caused by freeze-thaw or thermal cycling. It includes laboratory research done with the Inkom aggregate as duplication of the type of failure observed under field conditions. The first part also includes test observations on the amount of stripping in control and variant specimens subjected to freeze-thaw. Void or pore pressure as a failure mechanism is the subject of the second part. Reasons for postulating this type of failure are advanced.

Stripping Caused by Freeze-Thaw Cycling

Most research performed on debonding of asphaltic concrete is aimed at preventing failure. This research project began as an examination of one pavement near Pocatello, Idaho using Inkom aggregate (BK142-S) that had already showed stripping failure in an attempt to identify the failure mechanism. Duplication of the failure was essential before identification of the mechanism could begin. Because freezing in saturated portland cement concrete often causes failure, freeze-thaw cycles of 0 - 120 - 0 ^{o}F were chosen as a possible means of duplicating the field failure. Use of freeze-thaw is a reasonable simulation of field conditions in spring and fall when temperatures vary widely and pavements are most likely to be water-saturated.

Early testing by freeze-thaw cycling showed some stripping when the specimen was partially saturated with water, but no debonding was observed in dry specimens under similar conditions or when the specimens were saturated but not freeze-thaw cycled. A vacuum-saturation technique (see Appendix B) was developed immediately following these observations to completely saturate test specimens. Vacuum-saturated specimens subjected to 21 cycles of freeze-thaw showed the same type of failure as samples of pavement taken from the field. A freeze-thaw stripped specimen containing Inkom aggregate is shown with the actual pavement sample in Figure 1. In both cases, note the abundance of bare aggregate particles in the larger sizes. Appearance of the



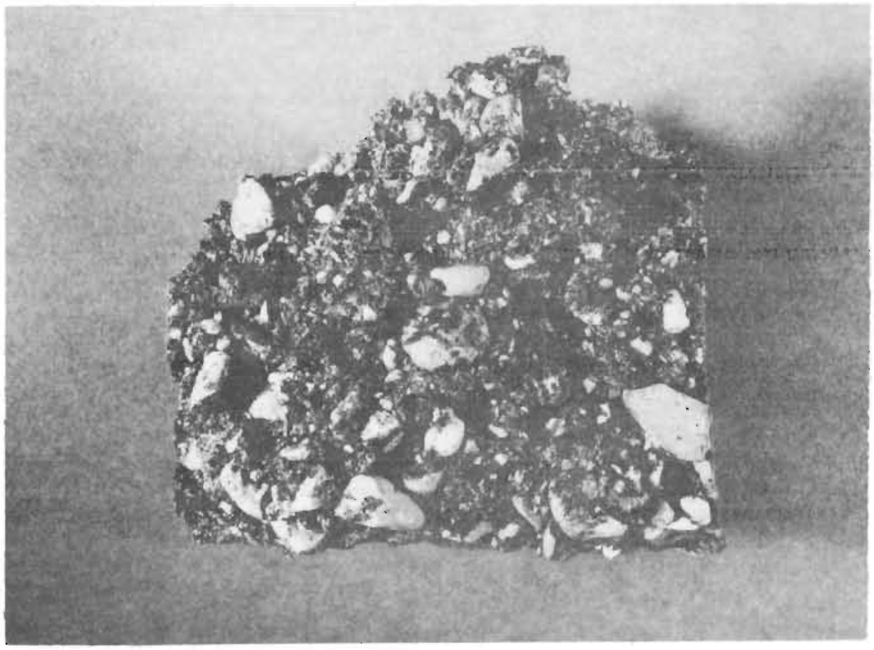


Figure 1. Top: Field Sample Showing Stripping in Actual Pavement.
Bottom: Laboratory Duplicated Sample Showing Stripping After Freeze-Thaw Test

outside of failed specimens is satisfactory with no indication of stripping, but structural strength of these specimens is very low.

Control Specimens

Having duplicated the field failure mode using freeze-thaw, it was decided to use this test to indicate the stripping potential of control specimens. Inkom aggregate was chosen as the main test aggregate (see Appendix A for gradation and description). Another aggregate, Washington No. 28, was used for a very limited number of specimens. Four asphalts, designated A, B, Ba and C (all 85/100 penetration), were used in combination with the Inkom aggregate.

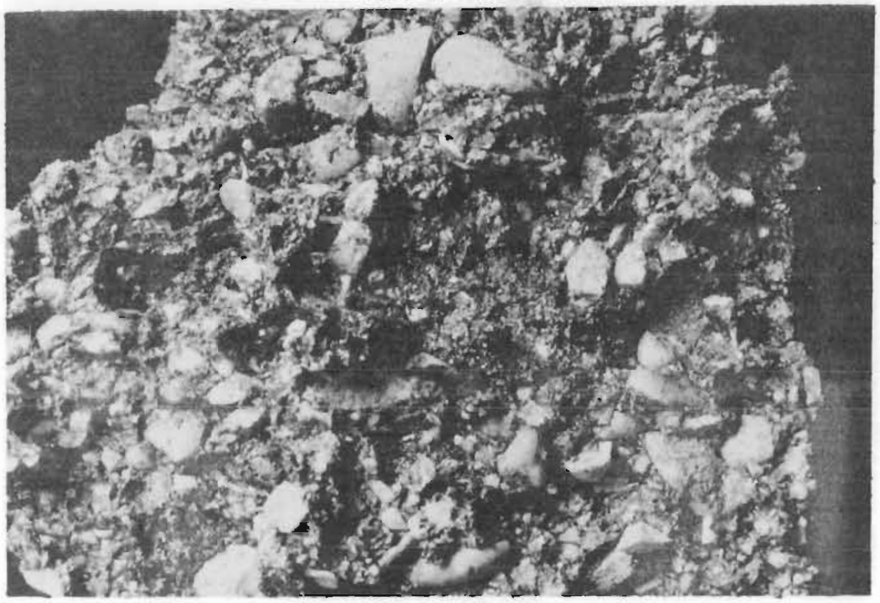
Five specimens using asphalt A, B and C and two specimens using asphalt Ba were made with Inkom aggregate at 4.94 percent asphalt content (aggregate basis). This asphalt content was the resultant average of extractions performed on pavement samples. All specimens received identical treatment during the mixing and compacting procedure (see Appendix B). After vacuum-saturation one specimen was broken open without cycling (0 cycles). The others were broken open at 3, 9, 15 and 21 cycles. Amount of stripping was visually estimated as percentage of total area believed to be bare aggregate.

It was found that the amount of stripping does increase proportionally to the number of cycles, but specimens having undergone only a few cycles of freeze-thaw still show significant stripping or adhesion failure. Close-up views of control specimens are presented in Figures 2 and 3. Note the many completely bare larger sized aggregate particles. In almost all cases a socket with very smooth sides was left where the aggregate particle came out. These sockets will be further discussed in the second part of this section.

Variant Specimens

Several variations from the control asphalt-aggregate combination were made and cycled in the same manner as the control specimens. The different or variant features are listed below with the first six incorporating Inkom aggregate:

I. All minus No. 200 aggregate dry sieved out.



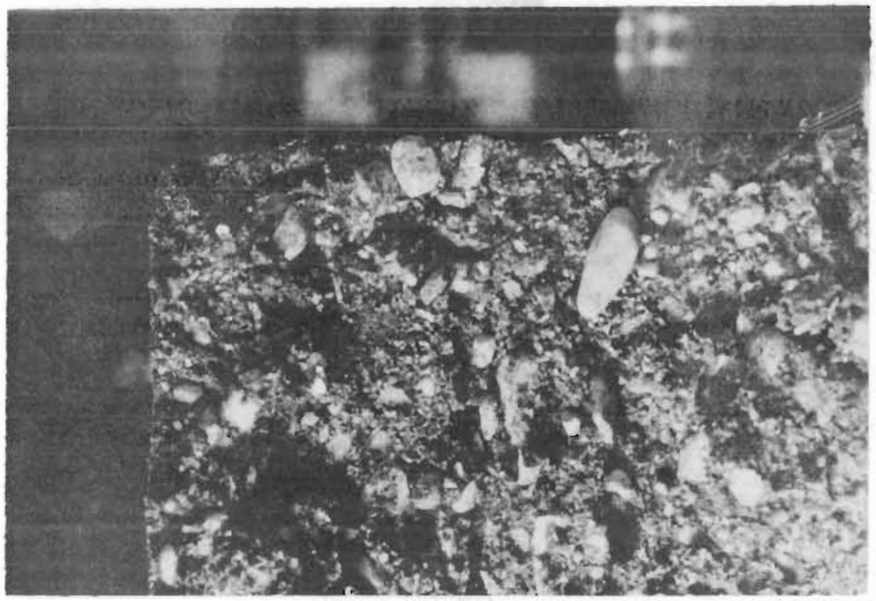


Figure 2. Close-up Views of Stripped Laboratory Specimens

Figure 3. Close-up View of Stripped Laboratory Specimen

2. All minus No. 200 aggregate washed out.
3. Same as 2, but used detergent soap in wash water.
4. Gradation was changed from 39 to 25 percent passing No. 10 sieve.
5. Four different chemical additives replaced 2 percent of the weight of asphalt cement.
6. Asphalt content was raised from 4.94 to 6 percent.
7. Washington No. 28 aggregate was used at design asphalt content of 6.2 percent of aggregate weight.

For most of the seven cases two specimens were prepared, cycled and then examined for stripping at 9 and 21 cycles.

These results indicate possible trends in the effects of these variations on stripping of the Inkom mixture. Good improvement in resistance to stripping was noted with one of the additives, but the others provided no significant improvement. Specimens made using the Washington No. 28 aggregate show a significant lack of stripping. This lack is probably due to the combination of (1) a higher asphalt content which fills the voids more completely and prevents full water saturation, and (2) the type of basalt aggregate surface itself, being porous and pitted, and chemically different as compared to the smooth-surfaced Inkom aggregate. These factors could result in better adhesion properties and lower stripping susceptibility.

There was no improvement in stripping resistance due to mixture changes under variations 1, 2, 3, 4 and 6. Variations 1, 2 and 3 are quite similar in that they all had reduced amounts of material passing the No. 200 sieve. All specimens were quite weak and porous due to the lack of fines acting as a filler. Removal of the fine dust in variation 1 did not appreciably decrease the severity of stripping. When washing or detergent washing accompanied the dust removal as in variations 2 and 3, stripping still occurred but there were fewer completely bare aggregate particles. Elimination of the thin film of dust that coats large aggregate particles appears to improve the coatability of the aggregate particle with asphalt, but would not significantly improve stripping resistance with the Inkom aggregate.

Reduction of the percentage of aggregate passing the No. 10 sieve in variation 4 had an effect which was similar to the effect of variation 1. The mixture was porous and weak, and significant stripping occurred.

Increasing the asphalt content in variation 6 had the effect of reducing the porosity or void ratio well below the usual values (1.5 to 4 percent vs.

previous values of 4 to 8 percent). This reduced the amount of water entering the specimen, keeping some specimen areas entirely dry. Significant stripping was still prevalent, but it was not found in these dry areas where the asphalt content within the specimen, due to variation, was high.

A Pore Pressure Failure Mechanism

The concept of pore pressure in porous materials causing failure is not new. T. C. Powers advanced a hydraulic pressure theory for tensile failure of portalnd cement concrete in 1945 (4) and pore pressure in soils is sometimes measured directly in a triaxial cell, but very little research has been performed concerning hydraulic void (pore) pressure in asphaltic concrete.

The idea that void pressure was causing failure came about as a result of a routine examination of freeze-thaw stripped mixtures under 7 to 30 power magnification. It was noticed that when test specimens were slowly pulled apart at warm temperatures, many bare aggregate particles left smooth-sided sockets as they came out of the mixture. Examination of these sockets through the stereomicroscope revealed that the asphaltic binder pulled cleanly away from the aggregate particle, making the socket a mirror image of the aggregate particle surface. In almost all sockets there were also seen small pinholes or void paths leading to the asphalt-aggregate interface from the asphalt binder matrix.

Based on these visual observations, a conception of the situation at a typical void is presented in Figure 4. This void situation is entirely hypothetical since it is not known how the voids and void paths are arranged. This representation is based on observations and accumulated knowledge of the asphaltic mixture.

In Figure 4 it is seen that water from the void paths in the asphalt binder matrix under hydrostatic pressure has entered the interface of the asphalt binder and the aggregate particle. Then the water under pressure proceeds through the interface, displacing the binder, until the aggregate surface is coated by a thin film of water. Observations of specimens from freeze-thaw cycle tests showed entire aggregate particles surrounded by water in a thin film.

Photographs of the failure sockets in specimens are presented in Figure 5. In particular, note the void pinholes and crevices in the sockets where interfacial water entry was made. Also note the texture of the walls which is about

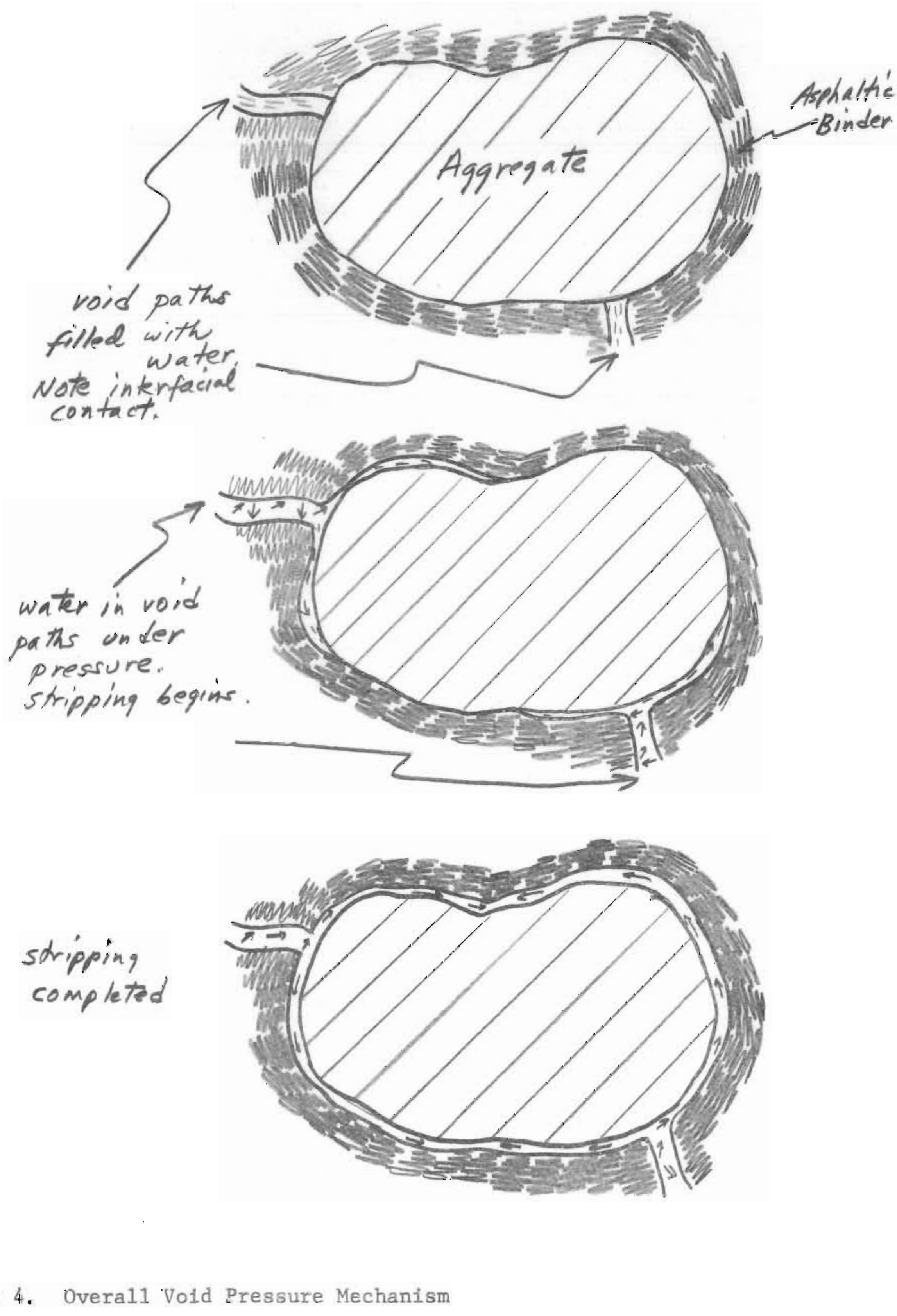


Figure 4. Overall Void Pressure Mechanism

Figure 5. Typical Binder Matrix Sockets Formed in Stripped Asphaltic Mixture Specimens

the same as the surface of the Inkom aggregate.

Similar sockets are observed in failed portland cement concrete specimens. The mode of failure for concrete is interfacial aggregate bond failure in these cases, and is thought to be produced by tensile forces. It is hypothesized that such tensile forces can be present in porous, saturated, compacted asphaltic mixtures and are responsible for stripping.

IV. VOID PRESSURE FAILURE MECHANISM

Typical asphaltic concretes are purposely designed with a small percentage of air voids to allow for differential thermal expansion of asphalt cement. Supposedly, these voids prevent the asphalt cement from being flushed onto the pavement surface during thermal expansion. Unfortunately these air voids may become saturated with water from rain, snow melt and even from vapor condensation due to water in the subgrade or subbase. A temperature rise after this saturation can cause expansion of the water trapped in the mixture voids, possibly resulting in significant void pressures when the voids are saturated.

Because asphaltic concrete is permeable, water could flow out of the void spaces under the pressure developed by the temperature rise and, in time, relieve the pressure developed. Thus the temperature rise causes pressure increase and some time-dependent pressure relief in the void pressure. Qualitative aspects of this theoretical mechanism are discussed in this section.

Conditions Assumed

Knowledge of the way in which water behaves in the void paths of compacted asphaltic concrete is very difficult to obtain. Gross measurements must be relied upon for data while the actual results are being caused by many micro-actions within the material. Consider the typical void situation represented in Figure 4 in the last section. After a temperature rise the following changes may be expected to occur in the saturated mixture:

1. Asphaltic concrete mixture expands and tends to increase the size of the voids,
2. Asphalt cement expands and tends to decrease the size of the voids, and
3. Water expands in the voids and tends to increase the size of the voids.

The separate effect of each change cannot be readily predicted. It appears however, that the mixture may not expand enough to accommodate the two fluid volume expansions. Considering gross measurements, a rough value for cubical thermal expansion of asphaltic concrete is $7 \times 10^{-5}/^{o}F$ (5) while that for water is $12 \times 10^{-5}/^{o}F$, a somewhat larger value. Expansion values for pure asphalt cement are about $3 \times 10^{-4}/^{o}F$ (5). Thus it may be assumed that there is not enough room available for expansion of the water and asphalt cement into mixture voids.

If the asphalt cement expands just enough to fill the expanding mixture then this will cancel out any change in the volume of voids. Volume expansion of the water will produce void pressure in the water and tensile stress in the mixture. The water under pressure attempts to flow out of the void area. If the permeability is high enough, then the water will physically leave the mixture; if not, then the tensile stress resulting from the pressure may break adhesion bonds and the water could flow around aggregates causing stripping.

The pressure inside a porous mixture would cause expansion of the mixture dictated by the tensile bulk modulus of the mixture. This slight volume strain would tend to lower the pressure. However, the quantitative aspects of how both volume strain and permeable flow would act together to dissipate some or all of the pressure developed was not investigated.

Temperature Induced Void Pressures

It was mentioned that temperature rises would cause water expansion in the voids, resulting in hydrostatic void pressures. Since the pressures remaining after some time may be difficult to calculate, the pressure changes may be computed to indicate the general magnitudes and maxima involved for comparison purposes with actual void pressures calculated later.

Assuming that the water in the voids is fully confined and that asphalt does <u>not</u> expand into the voids, then the void pressure would be a function of α_w, K_w (functions of temperature), n and T. The pressure change caused by a temperature change is:

$$Ho = (\alpha_w)(K_w)(\Delta T)(n)$$

where: Ho = pressure change resulting from temperature rise, psi,

α_w = coefficient of thermal expansion of water, $in^3/in^{3\,o}F$,

K_w = bulk modulus of water, psi,

ΔT = temperature increment, ^{o}F, and

n = void ratio, in^3/in^3 .

A plot of cumulative series of these pressures is shown in Figure 6 for several void ratios. The rise of the plot is due to the increase of K_w and α_w with temperature.

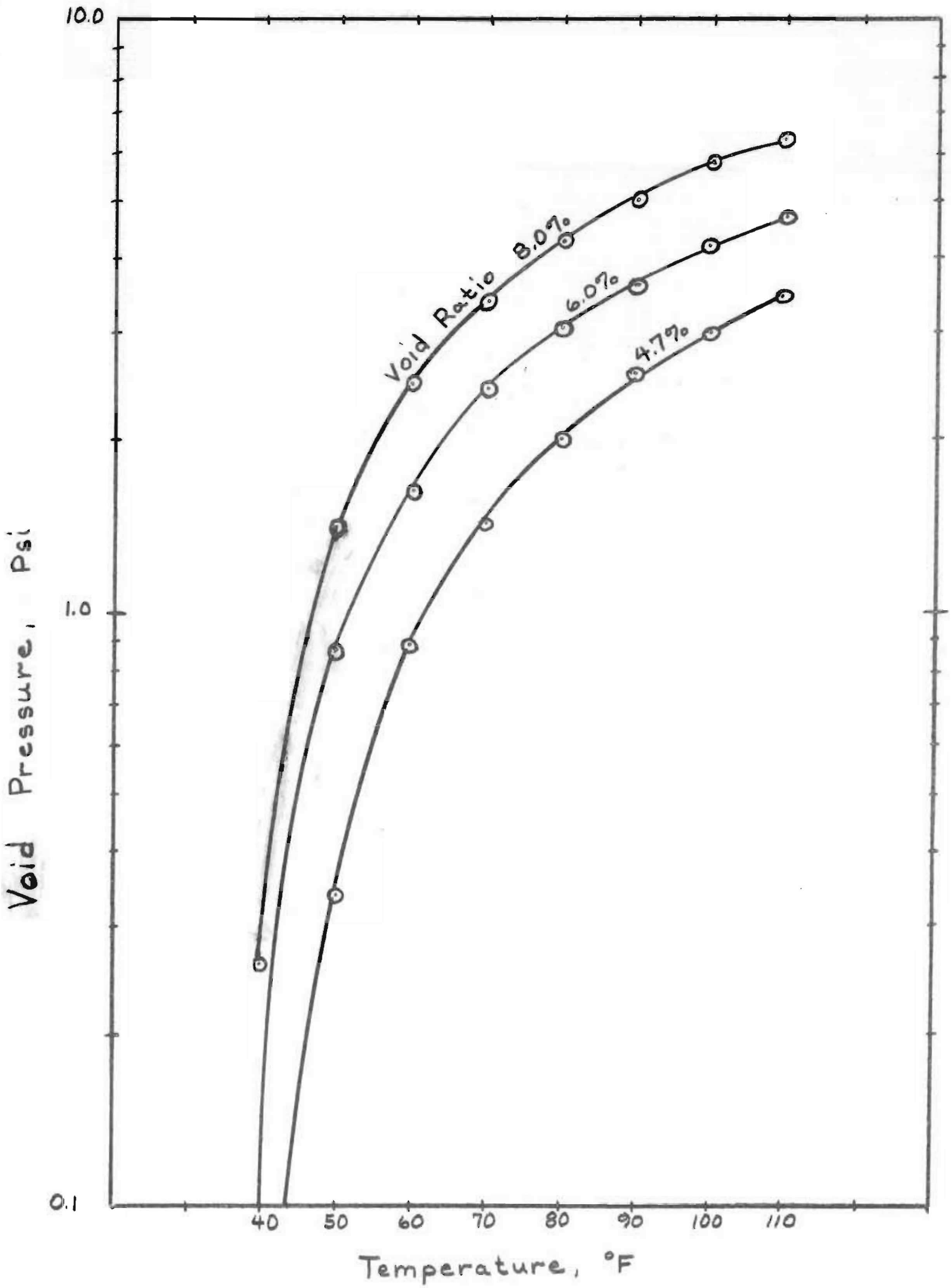


Figure 6. A Theoretical Trend of Void Pressure Induced by Temperature Change

V. INDIRECT MEASUREMENT OF VOID PRESSURE

An analysis of void pressures induced by thermal changes in saturated, compacted asphaltic concrete may be approached either by a direct or an indirect method. For example, a direct method would involve measurement of void pressure with a gauge or manometer on a specimen in a triaxial cell. A lateral confining pressure greater than the void pressure would be necessary to hold the membrane (that keeps the cell liquid from the specimen) in place. Such a method would, of necessity, measure pressure changes above the datum confining pressure surrounding the specimen during testing. Although such a test could be performed, it is possible that the void pressure values induced by a temperature change would be influenced by the confining pressure because of the viscoelastic nature of the asphaltic concrete mixture specimen.

The indirect method of void pressure analysis may be explained by considering the physical situation of an asphaltic concrete specimen being subjected to hydrostatic pressure in the void area. Such a pressure would cause the mixture to expand slightly. Volumetric expansion could be measured either by direct volume change or by measurement of axial strain and conversion to volumetric strain through use of Poisson's ratio. Volumetric strain of the mixture may also be stated as the result of a "tensile" pressure stress divided by the tensile bulk modulus of the mixture. Internal pressure can be calculated if both bulk modulus and volumetric strain are known. On the other hand, internal pressure can be calculated if axial modulus and axial strain are known. This will be subsequently discussed.

Axial Modulus

Determination of values for axial modulus required the examination of the effects of variables on the modulus. Asphalt type, void content, and temperature were postulated to be tha main variables. Temperature was specifically controlled at three levels while specimen voids were allowed to vary as a result of different compactive energies. Two asphalts coded A and C were used with the Inkom aggregate.

Bulk modulus cannot be calculated directly without knowing pressure and volumetric strain, but it may be found from Young's elastic modulus by the equation:

$$K = \frac{E}{3(1 - 2\mu)}$$

where: K = bulk modulus, psi,

E = elastic or axial modulus, psi, and

μ = Poisson's ratio.

Thus an experiment yielding data from which E is calculated may also be used to find values for K providing μ is also known.

Asphaltic concrete is a viscoelastic material and its response must be considered in this light. Generally if the application of the modulus constants is of very short duration, only the elastic constants will be necessary to describe the material's response. If the application is long term, then time dependent or viscoelastic constants are necessary. Modulus values in this case were to be used for calculation of stress (void pressure) induced by a temperature change. The exact manner in which void pressure changes with time is not known and therefore the time base for the modulus is also unknown. Consequently, an average value of E was calculated, determined from E at the instant of unloading (elastic) and after ten seconds of loading (viscoelastic) on cylindrical specimens. These two values were averaged to yield an E with time-dependent and with some time-independent properties.

The tensile, axial modulus values are shown in Figure 7. Actual data points were scattered and the plots represent average values determined by regression analysis.

Axial Strain Measurement

Measurement of strain of a saturated test specimen is not sufficient by itself because the specimen also has some volume change during a temperature rise. The desired volumetric strain is that due only to the void pressure. Therefore, the strain must be the additional strain of a saturated specimen as compared to a similar but dry specimen.

Axial strains were used with the axial modulus to calculate the internal

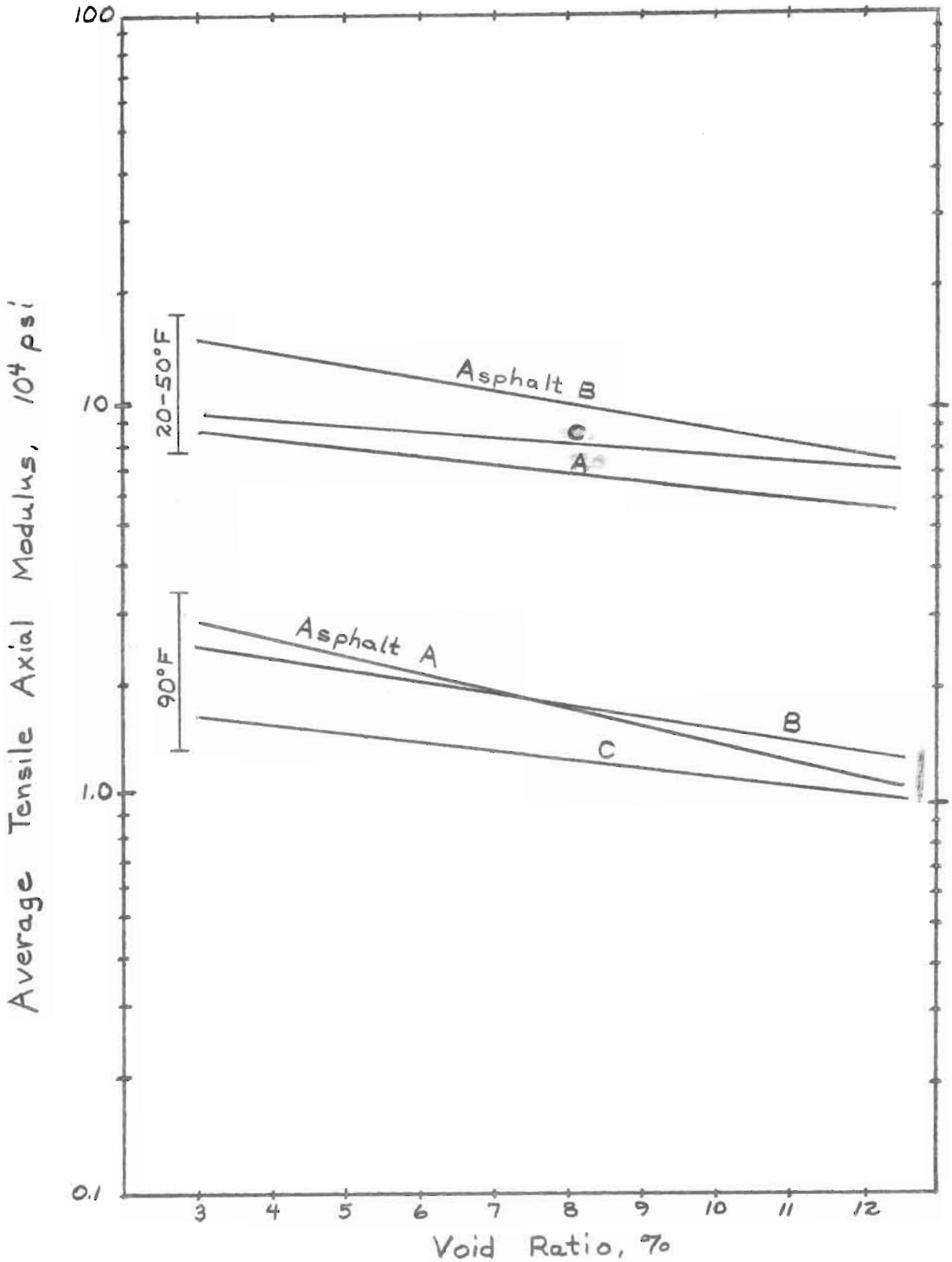


Figure 7. Average Tensile, Axial Modulus vs. Void Ratio

pressure by using the following relationship:

$$\varepsilon_v = 3\varepsilon\ (1-2\mu) \text{ where}$$

εv = volumetric strain, ε = axial strain, and μ = Poisson's ratio.
If the bulk modulus relationship is used, then

$$\text{Pressure} = \varepsilon_v K = 3\varepsilon(1-2\mu)\frac{E}{3(1-2\mu)}\ \text{, or}$$

$$\text{Pressure} = E\varepsilon\ .$$

Axial strains were measured through the use of waterproof strain gages attached to specimen pairs of the same void ratio and asphalt type and content. One specimen of the pair was kept dry by a wax coating; the other was vacuum saturated. Each pair was placed in a water bath and their axial strain differences were monitored as the water bath temperature was changed. Therefore, during a temperature rise, positive strain indicates that the saturated specimen shows height increase <u>relative</u> to the dry one. Void pressures are calculated using the previous equation with the axial modulus and axial differential strain at a particular temperature.

The strain gauge approach was used on four pairs of specimens made with Inkom aggregate and asphalt A, and three pairs made with asphalt C. All pairs of samples were tested at the same time in the laboratory water bath, and temperature increments of approximately 20°F were used from 10 to 110°F with twelve hours between the temperature increments (similar to the freeze-thaw test conditions). Strains for each pair were read at the end of each twelve hour period.

<u>Void Pressures</u>

Calculated void pressures are shown in Figures 8 and 9 for the two asphalt types used in making the specimens. Dashed lines are used for the two pairs that showed negative strains and therefore "negative" pressures.

One possible explanation for the negative differential pressures is the influence of the specimen area where the two layers of compaction meet. This area is centrally located perpendicular to the specimen's longitudinal axis and direction of strain measurement. It is possible in some specimens that this area is not as dense as the rest of the specimen. It absorbs more water and thus could contain large amounts of ice at the low temperatures

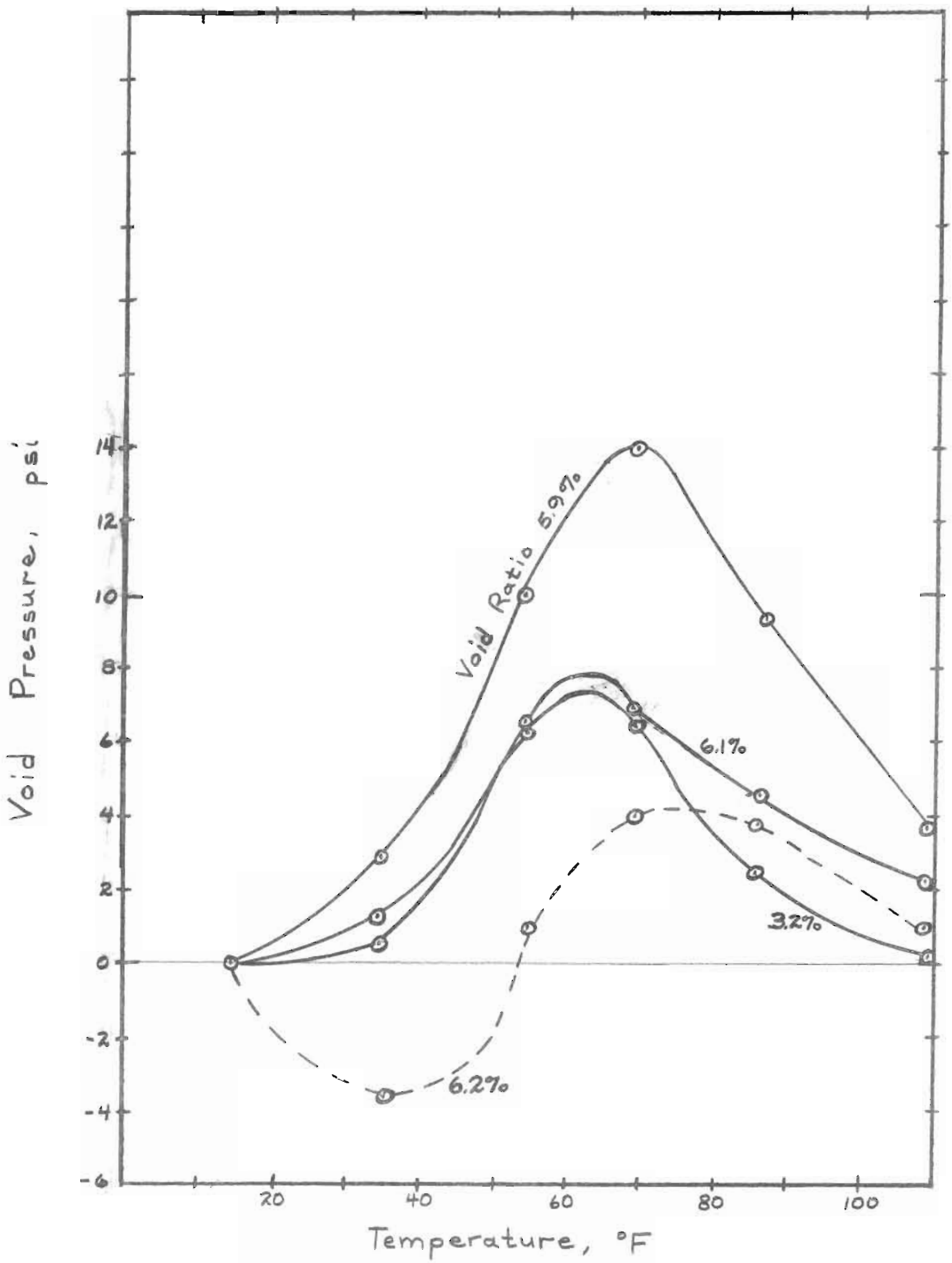


Figure 8. Void Pressure vs. Temperature, Asphalt A

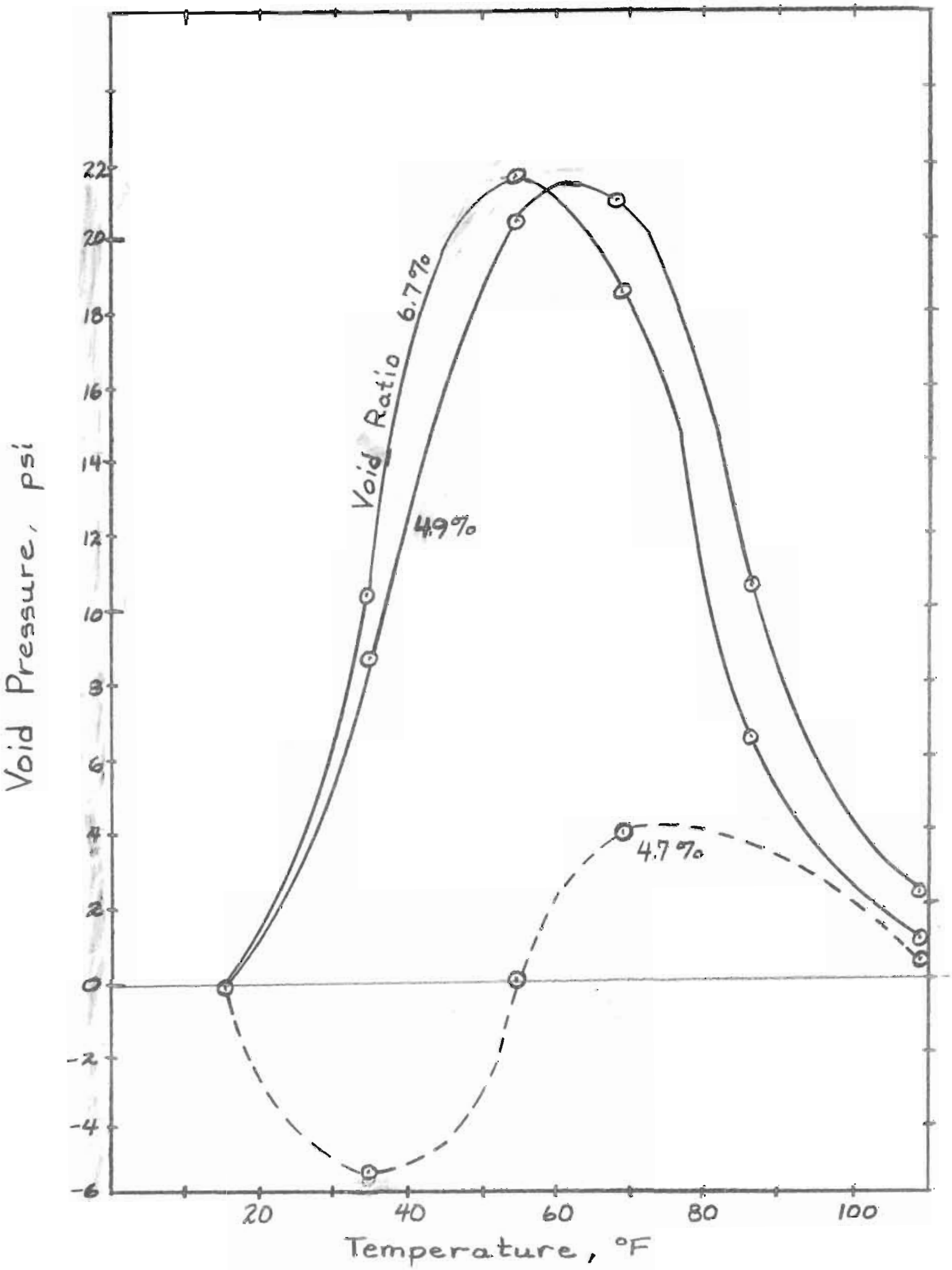


Figure 9. Void Pressure vs. Temperature, Asphalt C

used. At the base temperature the saturated cylinder would have been slightly expanded because of the ice expansion. When the ice melted, the asphaltic concrete might have crept back together, thus indicating a negative strain in this temperature range.

The positive differential pressures in Figures 8 and 9 indicate no trends based on void ratio, but actual pressures developed in the strain gauge test are generally higher but in the same range of values as those predicted in Section IV. All the void pressures peak in the 50 to 70°F range due to the influence of the temperature-dependent axial modulus. Although during the test the strains usually increased, the axial modulus decreases more rapidly, resulting in lower pressures. It is interesting to note that the peak pressure occurs near the mean of the temperature range usually experienced in asphaltic concrete pavement. This could indicate more stripping in this part of the temperature range.

VI. ANALYTICAL APPROACHES OF FAILURE MECHANISM

From a basic point of view it would be desirable to predict the stripping potential from basic physical constants of the asphaltic binder, aggregate and the resulting mixture. Surface energy or adhesion strength of the asphalt-aggregate interface could be equilibrated with maximum void pressure through quantitative use of an activating-resisting mechanism. Failure would be defined as the condition when activation or destructive forces exceed the resistive forces of the mixture bonds. Resistive forces could be made to exceed activating forces through experimentation with asphalt type, chemical additives, aggregate type and other factors influencing asphalt-aggregate interfacial adhesion strength.

The following are two possible methods for such an analysis based upon stripping caused by internal void pressure (internal tensile stress).

Method 1. Activating - Resisting Forces with Mixture Properties

Consider a compacted asphaltic mixture that is saturated. Assume that a void pressure is produced by thermal expansion and that this pressure creates an internal hydrostatic tensile stress in the mixture.

Let the activating force to cause stripping be equal to the product of the wetted surface area of the permeable voids and the void pressure.

Let the resisting force to inhibit stripping be equal to the product of the aggregate surface area which is contacted by the "voidless" asphaltic binder, and the adhesion strength of the binder-aggregate interface.

Stripping will occur if the activating force is greater than the resisting force.

Stripping could also occur even if the activating force is equal to the resisting force if a cyclic or fatigue void pressure loading is produced due to thermal change.

The activating force is

$$F_a = \frac{Aw}{V} \cdot P_v \text{ , where}$$

F_a = Activating force per cm^3 of mixture,

$\frac{Aw}{V}$ = Wetted surface area per cm^3 of mixture (a function of permeability and voids) and

P_v = Void pressure.

The Annual Report for Phase I (9) explains Aw/V and provides a graph for Aw/V vs. voids for the Inkom mixture.

The resisting force is

$$F_r = \frac{Ai}{V} \cdot P_a \text{ , where}$$

F_r = Resisting force per cm^3 of mixture,

$\frac{Ai}{V}$ = Surface area of bonded aggregate per cm^3 of mixture, and

P_a = Adhesion strength of binder-aggregate interface.

Ai/V can be determined from the gradation and specific gravity of the bonded aggregate. P_a can be determined from adhesion tests in the laboratory.

For the single cycle case, stripping failure occurs when $F_a > F_r$. The following is an example for the Inkom mixture.

Assuming 6 percent voids and using asphalt A, then Aw/V = 200 cm^2 per cm^3 of mixture (9). Assume P_v = 14 psi from Figure 8. Then

$$F_a = 200 \frac{cm^2}{cm^3} \cdot 14 \text{ psi } X \frac{1 \text{ in}^2}{6.45 \text{ cm}^2} = 434 \text{ lb. tension per 1 cm}^3 \text{ of}$$

mixture.

Ai/V is calculated from the Inkom gradation with an SG = 2.60 for the aggregate. Assume that all minus No. 200 aggregate particles in the gradation are mixed with the asphalt A to form a "voidless" asphaltic binder which bonds to the plus No. 200 aggregate particles. Therefore Ai/V = 39 cm^2 per 1 cm^3 of mixture. (Note: if the minus No. 200 aggregate particles were not a part of the binder but were actually a part of the bonded aggregate in the mixture then Ai/V would be about twice as large. Therefore the resisting force would be twice as large.).

P_a is estimated to be 60 psi at 70°F and at 80 micron film thickness of binder. The 70°F temperature is about the temperature where the maximum void pressure, P_v, was found (Figure 8). Then

$$F_r = 39 \frac{cm^2}{cm^3} \cdot 60 \text{ psi } X \frac{1 \text{ in}^2}{6.45 \text{ cm}^2} = 364 \text{ lb. tension per /cm}^3 \text{ of}$$

mixture.

Since $F_a > F_r$ then stripping will occur in a single cycle.

Suppose we examine F_a vs. F_r at higher and lower temperatures than 70°F. At 120°F., $P_v \doteq 1.5$ psi (Figure 8) and therefore

$F_a = (200)(2)(1/6.45) = 62$ lb. per cm^3 of mixture.

$P_a \doteq 8.5$ psi (9) and therefore

$F_r = (39)(8.5)(1/6.45) = 51.2$ lb. per cm^3 of mixture.

Thus $F_a > F_r$ at 120°F. and stripping would also occur in this temperature range.

For 40°F., $P_v \doteq 5$ psi (Figure 8) and therefore

$F_a = (200)(5)(1/6.45) = 155$ lb. per cm^3 of mixture.

$P_a \doteq 100$ psi and therefore

$F_r = (39)(100)(1/6.45) = 605$ lb. per cm^3 of mixture. Thus $F_a \ll F_r$ at 40°F. and stripping would not occur in this lower temperature range. Hence the critical temperature range seems to be the middle range from 50-85°F.

It should be emphasized again that stripping is fatigue-like in character, and even though $F_a = F_r$, stripping could occur if F_a is repeated in a cyclic fashion. This is hypothesized from test observations of the cyclic tests performed in this research.

Method 2. Activating - Resisting Hydrostatic Stress

In this method, stripping failure is assumed to depend only on the magnitudes of void pressure, tensile stress at the asphalt binder-aggregate interface and the adhesion strength of the interface. An activating-resisting relationship would be

$$F_a' \gtrless F_r' \quad \text{where}$$

F_a' = Activating stress and F_r' = Resisting strength.

$F_a' = P_v + T_h$, where

P_v = hydrostatic pressure in saturated voids, and

T_h = equivalent hydrostatic or isotropic tensile stress at interface.

$F_r' = P_a$ where P_a = adhesion strength at interface.

For a given compacted, saturated asphaltic mixture, F_r' is constant and the condition at verge of stripping at constant temperature would be

$$F_r' = F_a' = P_v + T_h .$$

The freeze-thaw test produces equal P_v and T_h values since hydrostatic or isotropic tensile stress conditions exist. For example, at 70°F. the Inkom mixture with asphalt C (see Figure 9) contains a void pressure, P_v, of 18 psi in the 5-6 percent void range. This condition also produces an isotropic tensile stress, T_h, at the interfacial area of 18 psi. Therefore F_a' = 18 + 18 = 36 psi. Stripping was also observed in a pure hydrostatic pressure test on the same mixture when P_v = 35 to 40 psi and T_h = 0. This indicates that $P_v + T_h$ could be equal to a constant, F_a', for a given mixture and leads one to believe that the sum of any test combination of P_v and T_h that equals 36 psi or greater should produce stripping in the mixture. Further tests to date have been inconclusive.

This particular method lends itself to both conditions of thermal stressing due to traffic. Thermal stressing would produce both P_v and T_h, and traffic stressing would produce an additional T_h.

Test observations indicate that thermal stressing is the primary effect and traffic stressing is a secondary effect in producing stripping. This is because severe stripping is readily produced in 8 ± cycles in the freeze-thaw test.

Either of the two methods would be possible means of predicting stripping susceptibility or resistance and consequently lend themselves more to the basic engineering approach. However, empirical evaluation based on outcomes of freeze-thaw tests, for example, would provide quicker results considering the pressing need for an acceptable test method now. Unfortunately the empirical evaluation test does not directly provide information as to what is happening within the mixture or as to the importance of mixture variables.

VII. CONCLUSIONS

The following conclusions are based on test observations to date.

1. Characteristics and severity of stripping in pavements incorporating "Inkom aggregate" can be duplicated with laboratory-made vacuum-saturated, compacted asphaltic mixture specimens exposed to cyclic freeze-thaw test generally in the range of 0° - 120° F.

2. Stripping is produced by internal surging of water pressure in the voids of the mixture. It is developed primarily through differential thermal expansion of asphaltic binder, asphaltic mixture and void water.

3. Stripped aggregate particles are surrounded by a film of water which has displaced the asphaltic binder. It is hypothesized that stripping will occur when there is contact between the aggregate surface and an initial path of void water. Initial paths in the binder sockets of stripped aggregate particles are always observed.

4. The freeze-thaw cyclic test can also be used as an evaluation test as well as a conditioning test. After cycling, laboratory personnel can slowly pull apart the tested specimens at 120°F. and visually observe the severity of stripping. The test is immediately more useful than analytical approaches but it does not provide basic mechanistic information.

5. Some chemical additives in the asphalt considerably reduce the stripping; some do not. Tests so far indicate no significant improvement in stripping resistance due to treating the aggregate by conventional methods or by washing the aggregate. A change of aggregate type, ie. Washington No. 28, produced mixtures which had only a small degree of stripping.

6. High asphalt content and mixture densities will help to keep the water out of the mixtures, but test observations and analytical implications indicate that voids must be no more than 2 percent. It is possible that pavement flushing will occur at the 2 percent or less voids.

VIII. BIBLIOGRAPHY

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APPENDIX A

AGGREGATE GRADATION AND DESCRIPTION

AGGREGATE GRADATION AND DESCRIPTION

The major aggregate used in this investigation is from Idaho Department of Highways source No. Bk - 142S, referred to as Inkom aggregate. The gradation for this aggregate is shown in Figure 10 (9).

This aggregate source from Southeastern Idaho contains 74 percent quartz, 5 percent montmorillonite, 5 percent mica (illite), 5 percent calcite and traces of iron oxide, dolomite and talc (3).

Appearance of the aggregate particles indicates that this source is an alluvial deposit. Many of the 1/2 and 3/8" particles are smooth surfaced with some of the larger particles having only one or two fractured surfaces from the crushing to gradation operation.

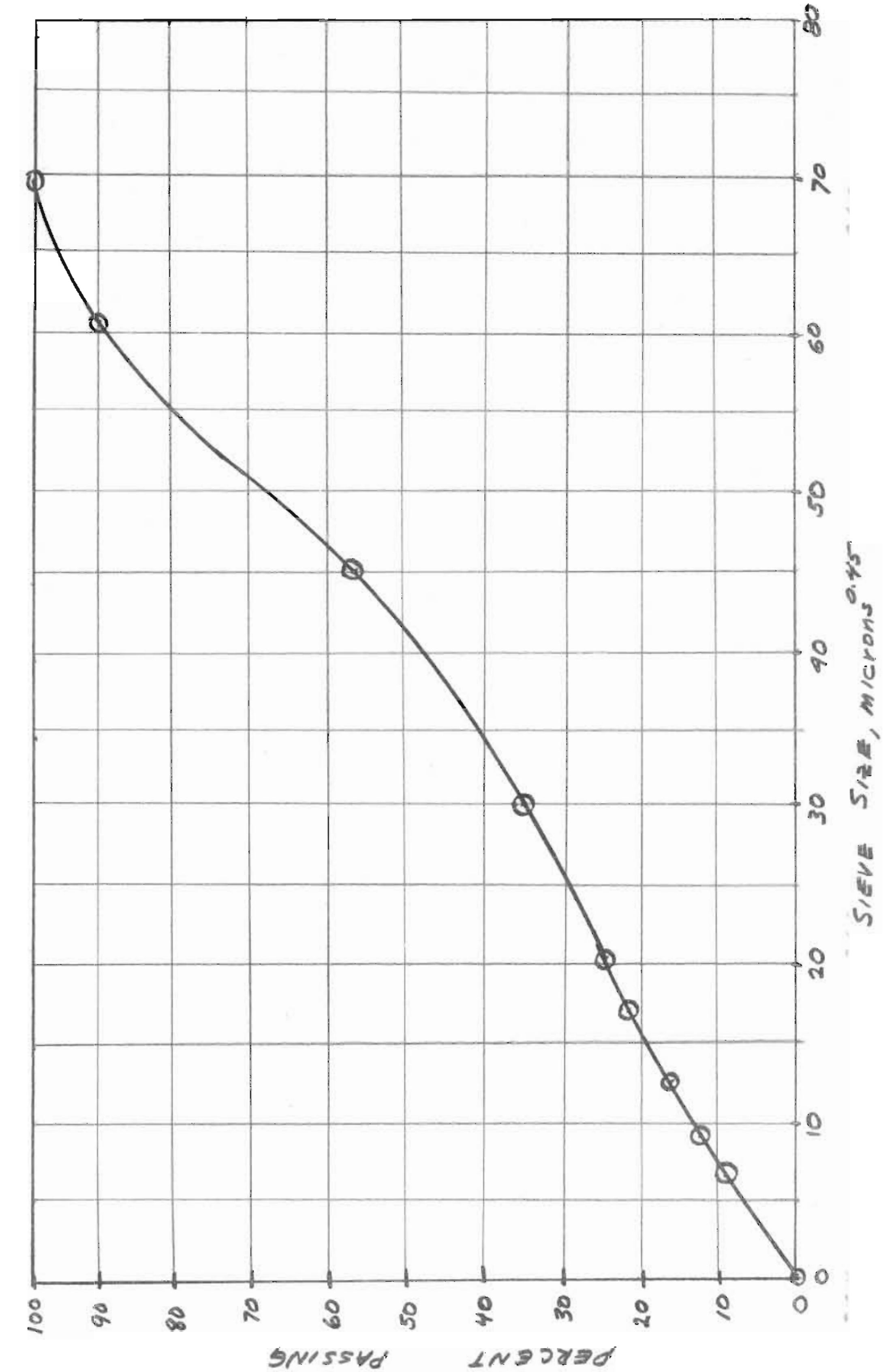

Figure 10. Inkom Aggregate Gradation

APPENDIX B

LABORATORY PROCEDURES

LABORATORY PROCEDURES

Laboratory procedures used in this research program were developed to meet the particular needs of the testing program. They are not accepted standards, but the procedure as outlined was carefully followed in each instance.

Mixture and Compaction

Asphalt used was of the 85/100 penetration grade. Aggregates were oven heated for 12 hours and asphalt for 2 hours to reach the mixing temperature of 325^{O}F. Mix time was two to three minutes in an electric whip-type mixer. Each batch of mixture was allowed to sit for 20 minutes in an oven to reach the compaction temperature. The compaction temperature varied from 200 to 300^{O}F with the higher temperature being used to produce specimens with fewer voids.

Compaction of the 4" by 5" high cylindrical specimens was accomplished with a kneading foot compacter by varying the number of blows and the dynamic load on each of two layers of mixture. Typical load-blow combinations at 200^{O}F are: 210 lb. and 35 blows on each of two layers, aimed to yield a void ratio of 8%, and 490 lb. and 70 blows on each of two layers, aimed to yield a void ratio of 4%. The above loads were applied to the mixture through a reciprocating foot of area 3.10 in^{2}. Similar combinations were used on the smaller 2.7" diameter by 5" high specimens used for the bulk modulus test.

After both layers were compacted, the specimens were leveled by applying a static load to both ends simultaneously, causing both ends to be parallel. The compacted specimen was then extracted from the mold.

Vacuum Saturation

A saturation technique was developed to permit more complete saturation of asphaltic concrete. First, the specimen was placed in a partial air vacuum of 4 to 5 inches of mercury for 30 minutes. This allowed the air voids to be reduced to the partial vacuum level from atmospheric pressure.

Second, water was introduced into the vacuum chamber at the same partial vacuum. The specimen remained in this partial vacuum for 30 minutes, allowing some water to soak into the voids. Third, while the specimen was still under water, the chamber was returned to atmospheric pressure. Because the void area was still at the partial vacuum level, water was driven into the void area by the difference of pressure from the outside. After 30 minutes the specimen was weighed to determine the amount of water driven into the voids.

Examination of the interior of specimens saturated in this manner showed a condition of maximum water penetration. Although this procedure does take 1-1/2 hours, the thoroughness of saturation has merit in that it may more closely simulate the condition of pavement after a year or two of environmental saturation.

Freeze-Thaw Cycling

High and low temperature settings were activated automatically by a set of clock-driven trippers in the environmental chamber used in the test. Trippers were set to turn the refrigeration unit on and off every four hours. This arrangement gave three cycles per day or 21 per week.

Specimens were staurated, placed in plastic bags, sealed and introduced into the chamber with a notation on date of removal. Upon completion of the set number of cycles, the specimens were slowly pulled apart longitudinally for display purposes. For convenience, the opening was done at $120^{\circ}F$.