Pressure-Induced Stripping in Asphaltic Concrete

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The purpose of this paper is to present a case for a significant stripping mechanism based on thermally induced pressure in the void water of saturated asphaltic-concrete mixtures. Although observations and measurements are obtained from asphaltic-concrete pavement and laboratory specimens representing typical asphaltic concrete in southeastern Idaho, the mechanism could also be prevalent in other states. Possible quantitative mechanisms are outlined for using the concept of activating void water pressure and resisting adhesion strength. It was found that void water pressures may develop to 20 psi under differential thermal expansion of the compacted asphaltic concrete and could exceed the adhesion strength of the binder-aggregate interface. Void water entrance to the interface was observed for all cases of stripping in the form of pinholes connected to void fissures. The voids (or fissures) are developed in the compaction phase of asphaltic-concrete production and are normally of the "correct" size to enable saturation. However, they are still of low enough permeability to enable the creation of void water pressures during temperature change. The problem discussed is one of the moderately weak asphalt-aggregate interface, depicting instances where static stripping tests are not illustrative of stripping potential and, consequently, some interfacial stressing is required.

•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.

Because 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, more recent investigations have been made on compacted asphaltic mixtures. It was found in previous Idaho experimentation that the use of compacted-mixture specimens in the testing program is a more direct approach than use of loose asphalt-aggregate mixtures. This is because internal void water or pore pressures, for example, can be created realistically in compacted-mixture specimens, producing forces that tend to strip the mixtures. It is hypothesized that these pressures, similar to those in actual Idaho pavements, either cause or appreciably accelerate stripping when the asphalt-aggregate interface is weak.

The objective of this paper, therefore, is to formulate the stripping mechanism as occurring in Idaho pavements by using test results obtained in the laboratory and observations made in the field.

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A recent study was made to determine the extent of stripping in Idaho (1). Pavement samples from throughout the state were taken and examined for stripping in both the +4 and -4 sieve size of 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. 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 2 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 minutes of the May 16, 1969, meeting (2). Testing in Arizona, California, Oregon, and Washington indicated that the Idaho aggregate would be suitable for use under present standards.

California research indicates that the Inkom aggregate is not susceptible to stripping as determined by conventional test methods on both loose and compacted mixtures (3). The immersion-compression tests showed a small (not serious) amount of stripping. It was attributed to the presence of about 5 percent montmorillonite in the aggregate fines.

Thus several agencies, acting independently, have shown the Inkom aggregate not to be susceptible to stripping. Yet the pavement constructed with this aggregate has shown extensive stripping failure. It was felt that a different approach was needed to more fully simulate field conditions because external or internal water action may not have existed in a realistic form in the conventional tests.

Freeze-thaw or temperature cycling as a method for measuring resistance to stripping has not yet been used for water-saturated asphaltic mixtures. However, Parr of the Michigan Department of State 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 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 the 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. If 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 because of 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 F. By raising the temperature of the cell to 95 F, they noted a corresponding pore pressure increase to 45 psi. Additional cycles of thermal rise and fall between these 2 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.

Research and testing to date on the causes of 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 subjected to freeze-thaw or other thermal cycling could be used in a test method for predicting stripping in asphaltic concrete.

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

STRIPPING CAUSED BY FREEZE-THAW CYCLING

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

Testing by freeze-thaw cycling showed extreme stripping under full saturation, some under partial saturation. No stripping was observed in dry specimens under similar conditions or when the specimens were saturated but not freeze-thaw cycled. A vacuum-saturation technique was developed 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 Figures 1 and 2. The photographs show an abundance of bare aggregate particles in the larger sizes. Appearance of the outside of failed specimens is normal with no indication of stripping, but cohesive strength of these specimens is very low.

Having seemingly duplicated the field failure mode by 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. It contained 74 percent quartz, 5 percent montmorillonite, 5 percent mica (illite), 5 percent calcite, and traces of iron oxide, dolomite, and talc (3). It was considered to be dense-graded. Another aggregate, Washington No. 28 (nonstripping), was used for a very limited number of specimens. Five specimens using asphalt cements A, B, and C and 2 specimens using another asphalt were made with Inkom aggregate at 4.94 percent asphalt content

Figure 1. Field specimen showing stripping in actual pavement.

Figure 2. Laboratory-duplicated specimen showing stripping after freeze-thaw cyclic test.

(aggregate basis). This asphalt content was the resultant average of extractions performed on actual pavement samples. All specimens received identical treatment during the mixing and compacting procedure. After vacuum-saturation, 1 specimen was opened without cycling (0 cycles). The others were opened at 3, 9, 15, and 21 cycles. Amount of stripping was visually estimated as percentage of total area believed to be bare aggregate.

The amount of stripping was found to increase proportionally to the number of cycles, but specimens having undergone only a few cycles of freeze-thaw still showed significant stripping or adhesion failure. The specimens showed many completely bare larger sized aggregate particles covered with a film of water. In almost all cases a socket with very smooth sides was left where the aggregate particle came out. Other bare aggregates could be pulled out easily and cleanly from their sockets.

Several variations from the control asphalt-aggregate combination were made and cycled in the same manner as the control specimens. In the following list of the different or variant features, the first six incorporate Inkom aggregate:

1. All minus No. 200 aggregate was dry sieved out,
2. All minus No. 200 aggregate was washed out,
3. All minus No. 200 aggregate was washed out by using detergent soap in the wash water and then washing out detergent,
4. Gradation was changed from 39 to 25 percent passing No. 10 sieve,
5. Four different chemicals were added at 2 percent of the weight of the asphalt cement,
6. Asphalt content was raised from 4.94 to 6 percent, and
7. Washington No. 28 aggregate was used at design asphalt content of 6.2 percent of aggregate weight.

For most of the 7 cases, 2 specimens were prepared, saturated, freeze-thaw 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 aggregate mixture. Good improvement in resistance to stripping was noted for one of the additives, but the others provided no significant improvement. Specimens made by using the Washington No. 28 aggregate showed a significant lack of stripping, probably because of the type of basalt aggregate surface itself, being porous and pitted, and chemically different as compared to the smooth-surfaced Inkom aggregate.

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, significant stripping still occurred but there seemed to be fewer completely bare aggregates.

Reduction of the percentage of aggregate passing the No. 10 sieve in variation 4 had an effect that 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 versus 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 internal variation, was high.

Thus, it appeared that "conventional remedies" were not generally successful for bringing the Inkom aggregate mixture up to the stripping resistance of a nonstripping mixture such as the one made with Washington No. 28 aggregate. Research work is continuing in Idaho on the specific problem.

Present testing has also shown that warm-cool thermal cycles, such as 40-110-40 deg, also produce or accelerate stripping in the same manner as described with the freeze-thaw cycling.

INDICATIONS OF VOID WATER PRESSURE

The idea that void water 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 shown in Figure 3. This void situation is entirely hypothetical because it is not known how the voids and void paths are arranged. This representation is based on observations and accumulated knowledge of the Inkom asphaltic mixture. It is hypothesized that water from the void paths in the asphalt binder matrix under hydrostatic pressure enters 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 aggregates surrounded by a film of water.

Photographs of an interfacial displacement and a resulting socket are shown in Figures 4 and 5. Visible are the void pinholes and crevices in the socket where interfacial water entry was made. The texture of the walls is about the same as the surface of the Inkom aggregate shown.

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 this stripping action.

TEMPERATURE-INDUCED VOID WATER PRESSURE

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, snowmelt, and even vapor condensation due to the rise of water vapor in the subgrade or subbase. A temperature rise after this saturation can cause expansion of the free 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 probably causes pressure increase and some time-dependent pressure relief in the void pressure. Qualitative aspects and measurements are discussed in this section.

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 on for data while the actual results are being caused by many microactions with the material. Consider the typical void situation shown in Figure 3. 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.

For gross measurements, a rough value for cubical thermal expansion of asphaltic concrete is 7×10^{-5}/deg F (5), while that for water is 12×10^{-5}/deg F, a somewhat larger value. Expansion values for pure asphalt cement are about 3×10^{-4}/deg 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.

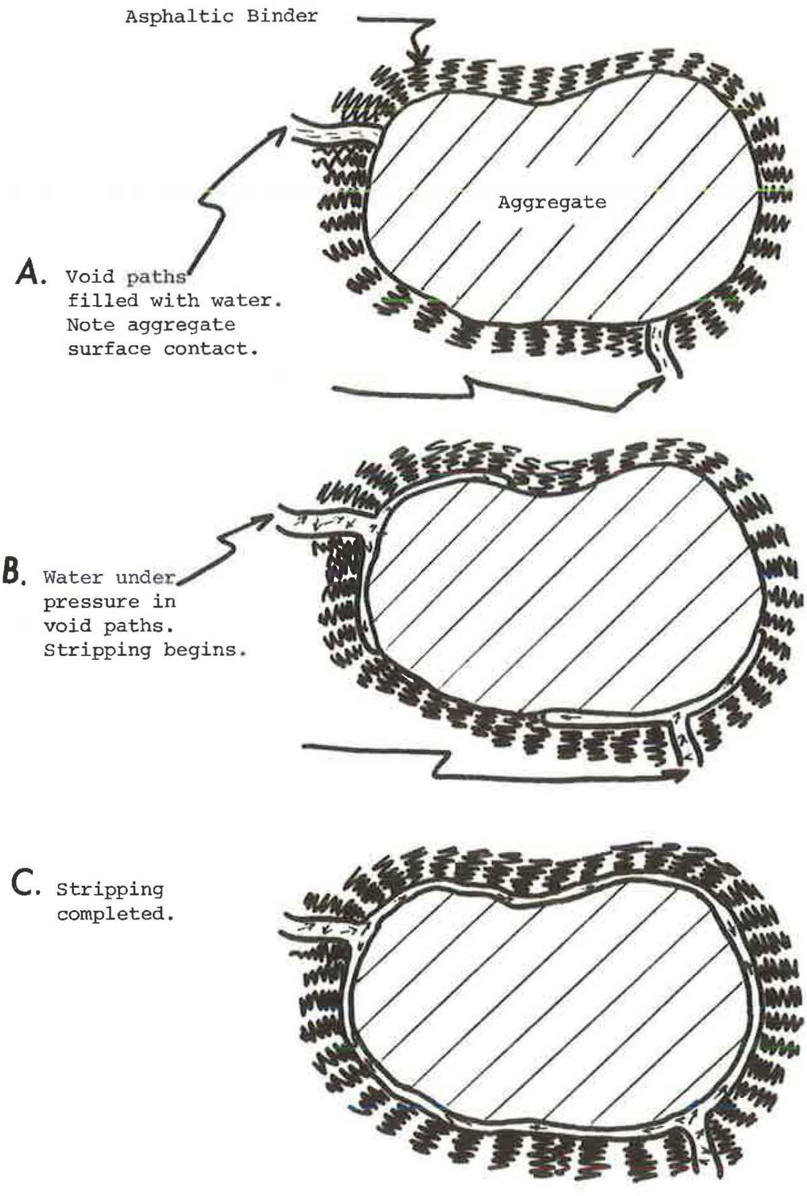

Figure 3. Hypothesized void pressure mechanism.

Figure 4. Typical binder-aggregate interfacial displacement.

Figure 5. Displacement socket in binder.

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, in contact with parts of the aggregate surface, 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 aspect of how both volume strain and permeable flow would act together to dissipate some or all of the pressure developed was not investigated.

An 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, if strains are small, internal pressure can be calculated if axial modulus and axial strain are known. This will be subsequently discussed.

Bulk modulus may be found from Young's elastic modulus by the following equation:

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

where

K = bulk modulus, psi;
E = elastic or axial modulus, psi;
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 10 sec of loading (viscoelastic) on cylindrical specimens. These 2 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 6. Actual data points were scattered, and the plots represent average values determined by regression analysis.

Axial strains were used with the axial modulus to calculate the internal pressure by using the following relationship:

$$\epsilon_V = 3\epsilon(1 - 2\mu)$$

where

ϵ_V = volumetric strain;
ϵ = axial strain; and
μ = Poisson's ratio.

If the bulk modulus relationship is used, then

$$\text{Pressure} = \epsilon_V K = 3\epsilon(1 - 2\mu)\frac{E}{3(1 - 2\mu)}$$

or

$$\text{Pressure} = E_\epsilon$$

The strain was considered to be the additional strain of a saturated specimen as compared to a similar but dry specimen.

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 costing; 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 has a height increase relative to the dry one. Void pressures were calculated by using the previous equation with the axial modulus and axial differential strain at a particular temperature.

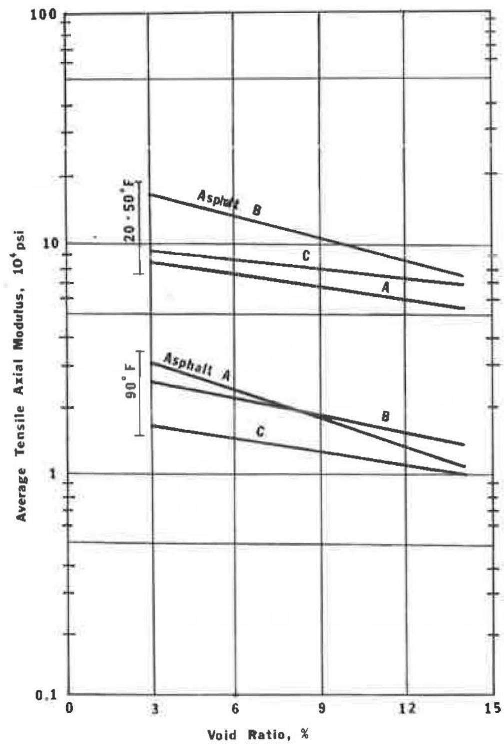

Figure 6. Axial moduli of compacted Inkom asphaltic mixture.

This approach was used on 4 pairs of specimens made with Inkom aggregate and asphalt A, and 3 pairs made with asphalts B and 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 12 hours between the temperature increments (generally similar to the freeze-thaw test conditions). Strains for each pair were read at the end of each 12-hour period.

Calculated void pressures are shown in Figures 7 and 8 for asphalts A and B used in making the specimens. Two pairs showed negative strains and, therefore, "negative" pressures.

One possible explanation for the negative differential pressures is the influence of the specimen area where the 2 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 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 shown in Figures 7 and 8 indicate no trends based on void ratio. All the void pressures peaked in the 50 to 70 F range because of the influence of the temperature-dependent axial modulus. Although during the test the strains usually increased, the axial modulus decreased more rapidly, resulting in lower pressures. It is interesting to note that the peak pressure occurred near the annual mean of the temperature range usually experienced in actual pavement. This could indicate more stripping in this part of the temperature range.

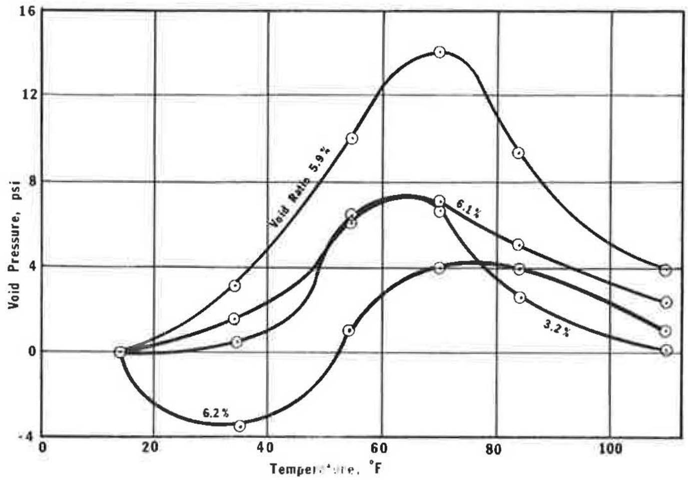

Figure 7. Void water pressure versus temperature for asphalt A.

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, the aggregate, and the resulting mixture. Surface energy or adhesion strength of the asphalt-aggregate interface could be equated to 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 3 possible methods for such an analysis based on stripping caused by internal void pressure (internal tensile stress).

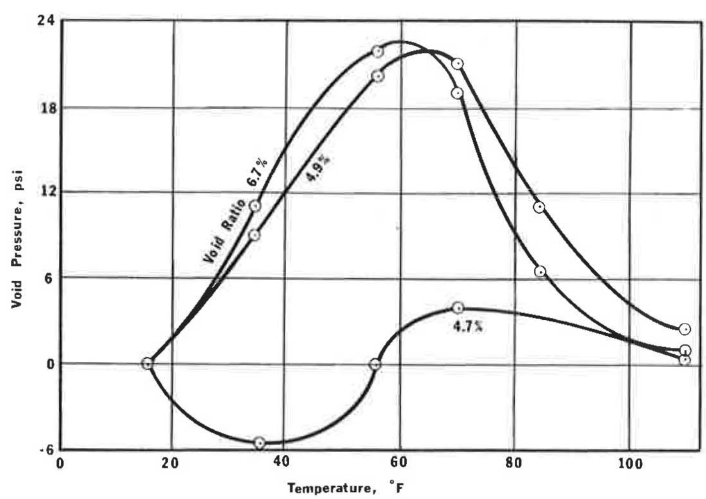

Figure 8. Void water pressure versus temperature for asphalt B.

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 that is contacted by the "voidless" asphaltic binder and of the adhesion strength of the binder-aggregate interface.

Stripping will occur if the activation 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 because of thermal change.

The activating force is as follows:

$$F_a = (Aw/V)P_v$$

where

F_a = activating force per unit volume of mixture;
Aw/V = wetted surface area per unit volume of mixture (a function of permeability and voids); and
P_v = void pressure.

The Appendix contains an explanation of Aw/V and a figure for estimating Aw/V versus voids for the Inkom mixture.

The resisting force is as follows:

$$F_r = (Ai/V)P_a$$

where

F_r = resisting force per unit volume of mixture;
Ai/V = surface area of bonded aggregate per unit volume of mixture; and
P_a = adhesion strength of binder aggregate interface.

Ai/V can be determined from the gradation and specific gravity of 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.

If there are 6 percent voids and asphalt A is used, then $Aw/V = 200\ cm^2/cm^3$ of mixture (Fig. 9). If $P_v = 14$ psi (Fig. 1), then

$$F_a = [200(cm^2/cm^3)]\ 14\ psi\ (1\ in.^2/6.45\ cm^2)$$
$$(16.4\ cm^3/1\ in.^3) = 7{,}100\ lb\ tension/in.^3\ of\ mixture$$

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 that bonds to the plus No. 200 aggregate particles. Therefore, $Ai/V = 39\ cm^2/1\ cm^3$ 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 (8). The binder in this case is considered to be asphalt plus the minus No. 200 filler. The 70 F temperature is about the temperature where the maximum void pressure, P_v, was found (Fig. 7). Then,

$$F_r = [39(cm^2/cm^3)]\ 60\ psi\ (1\ in.^2/6.45\ cm^2)$$

$$(16.4\ cm^3/1\ in.^3) = 5{,}950\ lb\ tension/in.^3\ of\ mixture$$

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

Suppose we examine F_a versus F_r at higher and lower temperatures than 70 F. At 120 F, $P_v \rightarrow 1.5$ psi (Fig. 7) and, therefore,

$$F_a = (200)(2)(1/6.45)(16.4/1) = 1{,}020\ lb/in.^3\ of\ mixture$$

$$P_a \rightarrow 8.5\ psi\ (\underline{8})$$

$$F_r = (39)(8.5)(1/6.45)(16.4/1) = 840\ lb/in.^3\ of\ mixture$$

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

For 40 F, $P_v \rightarrow 5$ psi (Fig. 7) and, therefore,

$$F_a = (200)(5)(1/6.45)(16.4/1) = 2{,}540\ lb/in.^3\ of\ mixture$$

$$P_a \rightarrow 100\ psi\ (\underline{8})$$

$$F_r = (39)(100)(1/6.45)(16.4/1) = 9{,}900\ lb/in.^3\ of\ mixture$$

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 observattions 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, the tensile stress at the asphalt binder-aggregate interface, and the adhesion strength of the interface. An activating-resisting relationship is as follows:

$$F_a' \gtrless F_r'$$

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 is

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

The freeze-thaw test produces equal P_v and T_h values because hydrostatic or isotropic tensile stress conditions exist. For example, at 70 F, the Inkom mixture with asphalt C (Fig. 8) contains a void pressure, P_v, of 18 psi in the 5 to 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.

Method 3—Fracture Surface Work

A possible method is generally outlined based on an analogy with fracture mechanics. Here one would equate the stored elastic energy (activation) to the fracture surface work required to strip or displace the binder-aggregate interface (resistance).

Let the stored elastic energy be

$$E_a = (\sigma^2/2E)$$

where

E_a = elastic energy per unit volume of mixture;
σ = internal equivalent tensile stress; and
E = elastic modulus of the mixture.

And let the fracture surface work be

$$E_r = \gamma(\Delta A/\Delta V)$$

where

E_r = displacement surface work per unit volume of mixture;
γ = fracture surface energy area; and
$\Delta A/\Delta V$ = area of interface displaced per unit volume of mixture, which would equal A_i/V as defined in method 1.

σ would need to be measured directly, or indirectly as indicated previously in the experimental results shown in Figures 7 and 8. E is an experimentally determined elastic modulus for the compacted saturated mixture. γ is an experimentally calculated fracture surface work term depicting the resistance of the asphalt binder to completely displace from the aggregate surfaces. All would need to be measured at the given temperature desired.

Displacement or stripping would occur if $E_a > E_r$ under one loading of σ. If there is a cyclic effect, then stripping would be possible if E_a were about equal to E_r as suggested in method 1.

Summary of Methods

The methods outlined would be possible means for predicting stripping susceptibility or resistance. Probably methods 1 and 3 would lend themselves more to a basic approach using engineering or scientific units. 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 especially in Idaho. Unfortunately the empirical evaluation test does not directly provide information as to what is happening within the mixture or as to the basic importance of mixture variables.

CONCLUSIONS

The following conclusions are based on test observations to the end of 1969:

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 0-120-0 F.

2. Stripping appears to be 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 that 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 or thermal 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. As observed via the cyclic test, 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. A change of aggregate type, i.e., the nonstripping Washington No. 28, produced mixtures that had only a small degree of stripping.

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Appendix

TEST AND ESTIMATING PROCEDURES

Water Permeability Tests

Falling-head permeability tests were performed on vacuum-saturated, compacted, asphalt concrete specimens. Voids in each specimen were varied.

Reference to the falling-head permeability test can be found in most soil mechanics books. We used 100 in. of head at the beginning of each test. This head was found to be insufficient to drive water through those specimens at voids lower than 4 percent; therefore, our results are repeated for specimens having voids greater than 4 percent.

Careful preparation of mixture specimens was necessary because it was found that permeability was sensitive to slight mixture variations. In the beginning of the tests, specimens from the same mixture produced permeabilities of 100 percent or more in difference at the same void content. Weight blending of aggregate sized fractions, close control of oven temperature, asphalt content, and compaction were all required to reduce this variation.

Specimens, 4 in. wide by 5 in. high, were thinly coated on their sides (except ends) with paraffin to fill in surface voids. Two or 3 coatings of thermosetting resin were applied following the paraffin coating. After the coatings hardened, the ends of the specimens were scarified in order to remove more dense and nontypical fatty spots of asphaltic mixture that sometimes occur during compaction.

Voids were calculated by water permeability obtained during a vacuum saturation process after the specimens were coated, scarified, and weighed. Each specimen was placed in a dry glass jar, and a vacuum of 4 in. of mercury was applied for 30 min. Distilled water was introduced into the jar, and all remained at the vacuum for a second 30 min. The jar and its contents were then released from the vacuum to atmospheric pressure and allowed to stand a minimum of 30 min. The water was always at a 2-in. minimum above the top of the specimen in the jar during the second and third 30-min steps. Distilled water was used. Specimens were weighed for calculating permeable voids prior to permeability testing. This was the standard vacuum saturation procedure for all specimens tested in thermal cycling as well.

After saturation, specimens were placed in a rubber boot. Attached to the bottom of the boot was a plate with inlet connected to the water supply from the falling head tube. Two uniform tension straps secured the top of the boot to the specimen. Usually 1½ ft of water in the 2-in. diameter falling tube were allowed to pass through each specimen before it was refilled and the test measurements started. Tests were run at room temperature, and distilled water was used. Specimens were weighed after permeability testing. Usually a gain or loss of 2 to 3 grams of pore water was noted.

A special study of asphaltic mixture permeability would require apparatus more conducive to the determination of the coefficient of permeability. A test setup including a back pressure saturator as reported by Vallerga (11) would be a method for achieving accurate coefficients of permeability by using a minimum number of specimens.

Coefficients of permeability were plotted versus mixture voids. The coefficients increased as voids increased: 5×10^{-5} cm/sec at 4.7 percent voids, 10×10^{-5} cm/sec at 5.5 percent voids, 20×10^{-5} cm/sec at 6.5 percent voids, 100×10^{-5} cm/sec at 7.5 percent voids, and $1{,}000 \times 10^{-5}$ cm/sec at 8.5 percent voids.

It was very difficult to obtain permeable voids for the mixtures much lower than 4.5 percent by using the laboratory-compaction apparatus (kneading compactor) without crushing the aggregate and without adding more asphalt than what was reported as design content. Consequently, a 4 to 5 percent void range may be the lowest encountered for these mixtures on the road. Field results show this to be true. One, therefore, can conclude that all 3 mixtures are permeable to water and could be saturated at some or all of the time while in service.

Wetted Surface Area

If the wetted surface area of the specimen's voids is considered, it is possible to use the coefficient of permeability and porosity (voids, percent/100) to calculate this wetted area. This wetted area may be significant in the mechanism of stripping or other failure under the influence of water.

The basic derivation of wetted surface area can be found in permeability sections of some soil mechanics books. We used the derivation of the Kozeny-Carman relationship as presented by Young and Warkentin (12).

The relationship is

$$k = \frac{C_S}{nT^2S^2} \times \frac{n^3}{(1 - n)^2}$$

where

k = coefficient of permeability, cm/sec;
C_S = pore shape factor (assumed to be 0.4);
γ = density of water, dyne/cm^3;
η = viscosity of water, poise;
T = tortuosity of pores (assumed to be $\sqrt{2}$);
S = wetted surface area per solid volume, cm^2/cm^3; and
n = porosity

For asphaltic concrete specimens, let

$$S = Aw/vs$$

where

Aw = wetted surface area or surface area of permeable voids, cm^2; and
Vs = volume of aggregate plus asphalt, cm^3.

Because $n = Vv/V$ and $V_S + Vv = V$, where Vv = volume of voids, cm^3, and V = total volume, cm^3, the relationship then becomes

$$K = (Cs/T^2)(\gamma/\eta)(V/Aw)^2n^3 \tag{1}$$

If we use constants of $C_S = 0.4$, $T = \sqrt{2}$, and $\gamma = 980$ dyne/cm^3 and $\eta = 8.9 \times 10^{-3}$ poises, a rearrangement gives

$$Aw/V = 1.48 \times 10^2(n^3/k) \tag{2}$$

Assuming that the basic derivation and assumptions of the Kozeny-Carman relationship are valid for permeability testing of asphaltic mixture specimens, then we can calculate the wetted surface area per unit volume of specimen from void and coefficient of permeability data. It is interesting to note that both voids and coefficient of permeability are required for the calculation.

There can exist certain unlimited combinations of n and k in Eq. 2 such that Aw/V will be a constant. In other words, as voids change the coefficient of permeability could change such that the overall effect produces a constant wetted surface area within the specimen. This, however, would be unusual over a wide range in voids, considering that the aggregate-asphalt arrangement is changed during changes in compaction.

Equation 2 shows in the limits that (a) as $n \to 1$, the mixture is essentially an open tube and the wetted surface area approaches the surface area of a specimen (4 in. diameter by 5 in. high) and equals approximately 0.40 cm^2/cm^3, and that (b) as n becomes very small, k also becomes very small and must exist if n exists as determined by vacuum saturation. In the comparison of n^3/k, the quantity then must approach zero as n approaches zero. Thus, Aw/V approaches zero.

The wetted surface area per volume, Aw/V, reaches a maximum for n (or voids) between $n = 0$ and $n = 1$. The actual n at maximum Aw/V is dependent on the characteristics of the asphaltic mixture and its compaction. The practicality of having an n less than the n corresponding to maximum Aw/V is not ascertained; however, theoretically if one compacts a mixture giving a small n below the critical value, then the wetted

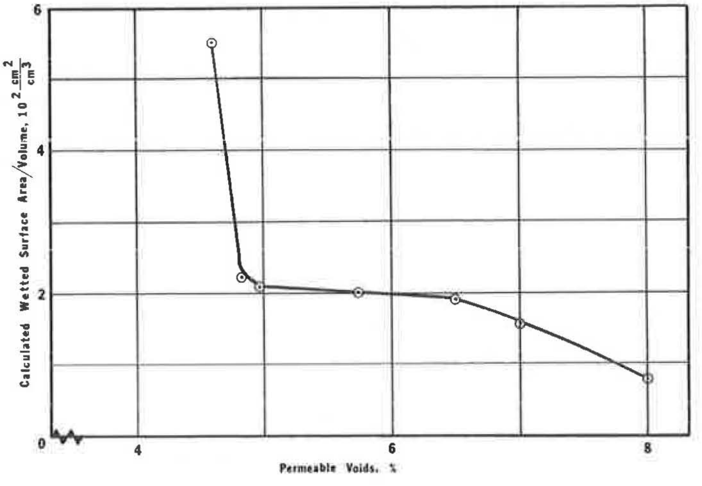

Figure 9. Wetted surface area versus voids.

surface area decreases because of pores closing up. This would be beneficial to the reduction of long-term stripping of asphalt from the aggregate. This critical value seems to occur around 3 percent voids.

Equation 2 generally is interpreted in the range of 3 to 10 percent voids. One can assume that, as voids increase for a given mixture, k will increase at a rate fast enough to make $(n^3/k)^{1/2}$ decrease. Thus, the wetted surface area should decrease. Although specimens at the higher voids will contain more quantity of water, the pore spaces are larger and provide less restricted flow. Several of the smaller pore paths combine to larger diameter paths that reduce the wetted surface area.

The wetted surface areas calculated from Eq. 2 are shown in Figure 9 as a function of voids. Aw/V data used in method 1 previously mentioned are obtained from this figure.