# TESTING FOR DEBONDING OF ASPHALT FROM AGGREGATES

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This paper presents the development of new apparatus and procedure for the measurement of stripping susceptibility of asphaltic concrete. In the new method a regular Hveem specimen [4-in. diameter by $2^1/_2$-in. height (101.6-mm diameter by 63.4-mm height)] was water saturated at 122 F (50.0 C). Then the specimen was subjected to repeated pore water pressure. The effect of the exposure on tensile strength was expressed as retained strength determined with a double punch procedure. The work showed that the method responded to variations in type of aggregate, cleanliness of aggregate, and asphalt content in a direction dictated by experience. Comparative results with the immersion compression test are presented.

• THE PHENOMENON of stripping has resulted in complete failure of a pavement within 2 weeks after opening to traffic and failure by loss of surface fines over a period of 2 years. To minimize the expense and inconvenience that result from rebuilding and resurfacing a damaged pavement, one has to expend much effort to define, measure, and predict the occurrence of asphalt stripping. The aim of this investigation was to develop an improved method for determining the susceptibility to stripping of asphalt concrete mixtures.

## FACTORS CONTRIBUTING TO STRIPPING

Early failure of asphalt pavement surfaces generally has been attributed to stripping because cracking from structural failure is not expected in the very early life of a highway. All stripping failures have been associated with the presence of water (1). Stripping or debonding implies a surface-to-surface separation; however, Schmidt (2) and others suggest that they are not necessarily adhesion failures but may be attributed to cohesion deficiency. It is not our intent to define or describe the mechanisms of the stripping failure but to list the factors that reportedly contribute to stripping occurrence. These are

1. Water,
2. Traffic,
3. Cool temperatures,
4. Low asphalt content,
5. Low asphalt viscosity,
6. High air void content,
7. Cleanliness of fine aggregate,
8. Coating on aggregate,
9. Composition of aggregate, and
10. Surface texture of aggregate.

The generalized mechanism of stripping is the rupture or displacement or both of the asphalt films that bind the aggregate in asphaltic concrete, which results in loss of tensile and abrasion resistance to traffic loads. The stresses that cause failure of the asphalt film are assumed to be water pressure and erosion caused by traffic or thermal cycles or both (3) on wet pavements.

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## TEST PROCEDURES

### Unconfined Compression

The best known procedure for determining the susceptibility to stripping of asphaltic concrete is the immersion compression test (ASTM D1075-54, AASHO T165-55). This procedure uses a numerical index for the complete and compacted design mixture to measure the loss of cohesion resulting from the action of water. Specimens are compacted by double-plunger action at a pressure of 3,000 psi (20.7 MN/m$^2$). The exposed specimens generally are soaked for 24 hours in a water bath held at 140 F (60.0 C). Testing is performed under unconfined compression at a deformation rate of 0.20 in./min ($8.3 \times 10^{-6}$ m/s) for a specimen 4 in (101.6 mm) high at a temperature of 77 F (25.0 C). The index of retained strength is obtained by dividing the strength of the exposed specimens into that of the control or dry specimens. Requirements on the value of the retained strength for acceptable mixtures vary from 55 to 75 percent.

### Marshall Compression

The Marshall test method has been used to determine resistance to stripping in much the same manner as the immersion compression test. Research done by Gallaway and Jimenez (4) showed that for certain aggregate-asphalt mixtures the results obtained by this procedure can be misleading.

### Confined Compression

The loss of strength or stiffness of asphaltic concrete because of the action of water and repeated stressing has been determined by Majidzadeh and Stander (5). In these works, triaxial compression with a repeated deviator stress was used.

### Split Cylinder Test

The most recent method for measuring stripping susceptibility of asphaltic concrete is based on the split cylinder test proposed by Lottman (6). A "retained strength" index is obtained from measurements of indirect tensile strength for specimens (or cores) 4 in. (101.6 mm) in diameter. Exposure is achieved by 12 freeze-thaw cycles of 0 to 120 to 0 F (-17.8 to 48.9 to -17.8 C) of water-saturated specimens. Strength testing is done at 55 F (12.8 C) with a deformation rate of 0.065 in./min ($0.3 \times 10^{-6}$ m/s).

These test procedures led us to conclude that there must be a simpler method to determine the susceptibility of asphaltic concrete to stripping or debonding.

## NEW TEST DESCRIPTION

Loss of cover material of asphaltic surfaces occurs during or soon after a cold rain on a trafficked pavement. The combination of water, lower temperature, and traffic stresses are the most critical at this time and cause raveling. Complete disintegration or cracking of the asphaltic layer that is caused by debonding of the asphalt from the aggregate is not a surface phenomenon as is raveling; therefore, more time is required for failures to manifest themselves. Visual examinations and research into these failures indicated that stripping or debonding was caused by the action of water.

The new debonding susceptibility test has 4 basic concepts.

1. Specimen size is to be approximately 4 in. (101.6 mm) in diameter by 2½ in. (63.5 mm) in height. However, field cores 3 to 4 in. (75 to 100 mm) in diameter and 2 to 4 in. (50 to 100 mm) in height can be tested.
2. Specimen is to be saturated because pore water pressure is to be used in the exposure portion of the test. It is believed that pore water pressure develops in the field.
3. The exposure of the specimen should be to only repeated pore water pressure at 122 F (50.0 C). The use of only pore water pressure simplifies the loading of different sized specimens. The temperature of 122 F (50.0 C) is near the value found by Jimenez (7) for saturated pavements in southern Arizona; this temperature reduces asphalt viscosity and thus resistance to debonding. The use of a 122 F (50.0 C) temperature rather

than a "cold rain" temperature was based on the thought that the loss of surface material in raveling was due to a brittle failure rather than an adhesive failure that related directly to stripping or debonding.

4. The strength test should be a simple, repeatable tension test. The test temperature for strength should be 77 F (25.0 C).

## Strength Test

The effects of stripping or debonding asphalt from the aggregate of asphaltic concrete should be most easily detected by tension testing. Because a direct tension test is not practical, some simple indirect tension test such as the cohesiometer, Marshall, split cylinder, or double punch test was considered. Prior experiences with the cohesiometer and Marshall tests were not particularly good and thus were not used. The work of Lottman (6) and others suggested the split cylinder procedure. However, the work reported by Fang and Chen (8) with the double punch test appeared most promising.

In the double punch test, a cylindrical specimen is centrally loaded on both the top and bottom surfaces with cylindrical steel punches. The penetration of the cones that develop between the punches serves to split the specimen along the weakest radial plane. Preliminary work with the split cylinder and double punch tests was carried out with 2 mixtures of asphaltic concrete compacted by vibratory kneading compaction (10). The split cylinder specimens were loaded through steel bars with a width of 1 in. (25.4 mm); the double punch test used punches 1 in. (25.4 mm) in diameter. Table 1 gives the repeatability of the 2 test methods and the relationship between the stresses obtained for 2 aggregate gradations. The specimens were compacted by vibratory kneading and were tested in triplicate (14). The data indicated that the double punch test had better repeatability than the split cylinder and that the stresses obtained by the 2 procedures were identical to each other because the value of the slope coefficient was almost 1.

Observations of these 2 test methods indicated that the double punch test was simple in operation and that stress analysis did not have to be corrected for flattening of the load surface as it would have in the split cylinder test.

## Specimen Size

Other preliminary work with the double punch test was concerned with the size of specimen and punch diameter. Specimens were compacted by vibratory kneading and tested in triplicate at 77 F (25.0 C) with a deformation rate of 1 in./min (25.4 mm/s) (15). Specimen diameter size varied from 2 to 4 to 6 in. (50.8 to 101.6 to 152.5 mm) and height size from 2 to 4 to 8 in. (50.8 to 101.6 to 203.2 mm); punch diameters were from $^3/_8$ to $^5/_8$ to 1 in. (9.5 to 15.9 to 25.4 mm). The results of this testing are given in Table 2. The data indicated the following:

1. For constant specimen height, the tensile stress decreased as the ratio of specimen diameter to punch diameter (D/d) increased; and
2. For constant specimen height and D/d ratio, the tensile strength decreased as specimen diameter decreased.

## Specimen Saturation

A review of the literature indicated that vacuum saturation of asphaltic concrete specimens is a rather simple and effective means of filling the voids with water. A limited investigation was done to determine the duration and amount of vacuum needed to saturate a specimen. Work was centered on a mixture with good resistance to the action of water and specimens compacted with the California kneading compactor. A submerged specimen could be saturated fully with distilled water with a vacuum of 20 in. of mercury ($67.6 \times 10^3$ N/m$^2$), held for 5 min at this pressure, and then 5 min at atmospheric pressure for a blotting effect. The temperature of the water was that of the laboratory—about 75 F (24.0 C).

Double punch tensile tests performed on control and vacuum-saturated specimens indicated no adverse effect or loss of strength caused by the vacuum-saturation procedure. However, later work with a more water-susceptible mixture and with vacuum

**Table 1. Tensile stress by indirect methods.**

| | | Split Cylinder[a] | | Double Punch | |
|---|---|---|---|---|---|
| Deformation (in./min) | Test Temperature (deg F) | Stress (psi) | Coefficient of Variation (percent) | Stress (psi) | Coefficient of Variation (percent) |
| Dense-Graded Aggregate | | | | | |
| 0.05 | 75 | 40.8 | 13.5 | 49.9 | 11.8 |
| 0.50 | 75 | 105.9 | 13.6 | 93.4 | 14.2 |
| 1.00 | 75 | 113.9 | 10.6 | 116.4 | 2.4 |
| 0.065 | 55 | 120.2 | 13.7 | — | — |
| Gap-Graded Aggregate | | | | | |
| 0.05 | 75 | 70.9 | 14.5 | 65.0 | 10.8 |
| 0.50 | 75 | 140.5 | 5.0 | 133.2 | 5.0 |
| 1.00 | 75 | 176.5 | 10.0 | 161.9 | 2.1 |
| 0.065 | 55 | 167.6 | 14.7 | — | — |

Note: 1 in./min. = $41.5 \times 10^{-6}$ m/s. 1°C = $\frac{5}{9}$ (°F - 32). 1 psi = $6.89 \times 10^3$ N/m$^2$.

[a]For dense-graded aggregate, $S_{S.C.} = 1.14\ S_{D.P.} - 12.2$ and $r^2 = 0.94$. For gap-graded aggregate, $S_{S.C.} = 1.07\ S_{D.P.} + 0.8$ and $r^2 = 0.98$.

**Table 2. Effects of specimen and double punch size on tensile strength.**

| Specimen Size (in.) | Punch Diameter (in.) | Tensile Strength (psi) | Sample Standard Deviation | Coefficient of Variation (percent) |
|---|---|---|---|---|
| 6 by 2 | 1 | 145 | 11.4 | 7.9 |
| 4 by 2 | 1 | 150 | 5.6 | 3.7 |
| 2 by 2 | 1 | 284 | 22.2 | 7.9 |
| 6 by 2 | $\frac{5}{8}$ | 82 | 4.5 | 5.6 |
| 4 by 2 | $\frac{5}{8}$ | 74 | 2.1 | 2.8 |
| 2 by 2 | $\frac{5}{8}$ | 130 | 15.0 | 11.6 |
| 4 by 2 | $\frac{3}{8}$ | 47 | 1.7 | 3.7 |
| 2 by 2 | $\frac{3}{8}$ | 52 | 12.5 | 24.1 |
| 4 by 4 | $\frac{5}{8}$ | 42 | 0.6 | 1.4 |
| 4 by 8 | $\frac{5}{8}$ | 22[a] | | |
| 4 by 8 | 1 | 40[a] | | |

Note: 1 in. = 25.4 mm. 1 psi = $6.89 \times 10^3$ N/m$^2$. 1°C = $\frac{5}{9}$ (°F - 32).

[a]Not a tensile failure.

**Figure 1. Disassembled saturation and stressing chamber.**

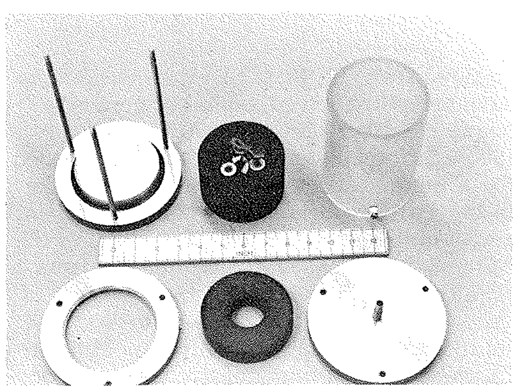
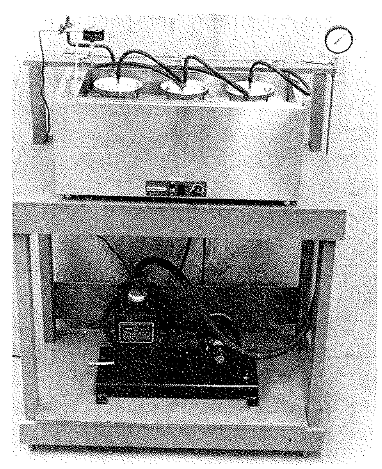


**Figure 2. Water bath and saturation setup.**



saturation at 122 F (50.0 C) indicated that this combination of exposure can cause a loss of tensile strength of as much as 20 percent of the dry specimen.

### Specimen Stressing

The desire to simulate traffic loading conditions that cause debonding resulted in using repeated pore water pressure for specimen exposure. This stressing or conditioning followed the vacuum saturation of the specimen at 122 F (50.0 C). The saturation chamber containing a submerged specimen was fitted with a rubber-like annulus that was submerged but not in contact with the specimen. A sinusoidal loading was applied to the annulus that caused a hydraulic pressure varying from 5 to 30 psi ($34.5 \times 10^3$ to $217.0 \times 10^3$ N/m$^2$) at a rate of 580 times/min. Because the saturation chamber was clear plastic, the muddying of water and surging of loose sand particles could be observed as the stressing of a specimen continued. Repeated pore water pressure and surging of sand particles within the saturated specimen approached the stressing that a rain-soaked pavement receives from moving traffic loads.

## DEBONDING TEST PROCEDURE

Because hot, water-saturated pavements may be most susceptible to debonding failure when subjected to dynamic loading (traffic), a duplication of the physical state of such pavements was attempted in the laboratory by both saturating and stressing the specimen. Given a 2½- by 4-in. (63.5- by 101.6-mm) cylindrical asphaltic concrete specimen, the following basic procedure should be used to determine its resistance to debonding: (a) saturate specimen; (b) subject it to cyclic stressing; and (c) test it for strength.

### Saturate Specimen

Previous testing has shown that the following procedure will yield a high degree of saturation (99 to 100 percent) in a short time. Research has shown that, during the summer months in the desert Southwest, temperatures in saturated asphaltic concrete pavements seldom fall below 122 F (50.0 C). Water in a bath used for saturating specimens was maintained at 122 F (50.0 C) to a depth of approximately 8 in. (200 mm); distilled water was preferred. The following step-by-step procedure to full saturation should be used:

1. Place specimen in the plexiglass saturation-stressing chamber (Fig. 1);
2. Place chamber in hot-water bath, cover specimen with about 2 in. (50 mm) of hot water, and secure lid on chamber (Fig. 2);
3. Allow specimen to stand in hot water for approximately 15 min;
4. With spigot at base of chamber closed, connect vacuum hose to top of chamber and apply vacuum pressure (20 in. of mercury) ($67.6 \times 10^3$ N/m$^2$) for 5 min; and
5. Release vacuum and let specimen stand an additional 30 min in hot-water bath to bring specimen to bath temperature.

### Cyclic Stressing

This method was the result of previous work and was designed so that the specimen would not be loaded directly. Loading was accomplished through a layer of water ¼ to ½ in. (6 to 13 mm) deep between the rubber annulus and the top of the specimen. A cyclic load operating at 580 rpm was used to generate pressures within the water-saturated specimen ranging from 5 to 30 psi ($34.5 \times 10^3$ to $217.0 \times 10^3$ N/m$^2$)—a range comparable to that expected in saturated pavements under traffic. The following stress procedure should be used:

1. With the chamber (and specimen) still in the hot-water bath, remove the vacuum-tight lid, replace it with a stress-ring lid, and secure this lid tightly by hand by turning the wing nuts;
2. Place the Flexane rubber annulus that is 1 in. (25 mm) thick into the chamber (under the water in the chamber) and carefully release all entrapped air from beneath the annulus;

3. Adjust the annulus until it is perpendicular to the cylindrical axis of the chamber and push it slowly down until approximately ¼ to ½ in. (6 to 13 mm) of water remains between its bottom surface and the specimen's top surface;
4. Remove the chamber from the hot-water bath and quickly place it in proper position on the vibratory kneading compactor table;
5. Lower the loading apparatus carefully until the foot 4 in. (101.6 mm) in diameter makes contact with top of annulus. Then continue lowering crossbar until annulus and water support the weight of the loader;
6. Set timer to the time required to obtain the desired number of load repetitions and activate the electric motor; and
7. After stressing time has elapsed, raise loader, remove chamber from compactor, remove stressing ring lid and annulus, and remove specimen from chamber and place it in a 77 F (25.0 C) water bath for a minimum of 45 min.

Figure 3 shows repeated pore water pressure stressing.

### Strength Test

After stressing and cooling, the tensile strength of the specimen was determined by the double punch test, which consists of loading with two steel rods (punches) 1 in. (25.4 mm) in diameter centered on both top and bottom surfaces of the cylindrical specimen.

If the specimen is to be tested while it is fully saturated, it should be taken directly from the 77 F (25.0 C) water bath and tested immediately. If a lesser degree of saturation is required, the specimen should be removed from the water and allowed to dry for a time before testing. When the specimen has reached the desired state of saturation, the following test procedure should be used:

1. Center specimen on bottom punch by using wooden centering blocks;
2. Lower the test machine head until the upper punch just touches the upper surface of the specimen; and
3. Set head speed to 1.0 in./min ($41.5 \times 10^{-6}$ m/s) and begin loading.

Figure 4 shows a specimen being tested.

Maximum load registered by the testing machine was considered the failure load. Tensile strength was calculated by using the following equation:

$$\sigma_t = \frac{P}{\pi(1.2bH - a^2)}$$

where

$\sigma_t$ = tensile stress, in pounds per square inch;
P = maximum load, in pounds;
a = radius of punch, in inches;
b = radius of specimen, in inches; and
H = height of specimen, in inches.

## MATERIALS FOR PROCEDURAL APPRAISAL

### Aggregates

Three general aggregate blends were selected to represent best, average, and worst.

1. The limestone aggregate was selected as representative of the best aggregates because it resists debonding well.
2. The Tucson aggregate was selected as representative of average aggregates from dry washes and pits because it is typical of the aggregate used for asphaltic concrete in the Tucson area.
3. The Holbrook aggregate was selected as representative of the worst aggregates because it is most susceptible to debonding.

Aggregate characteristics are given in Table 3. There was an inconsistency between sand equivalent value and performance; limestone aggregate had a lower (worse) sand

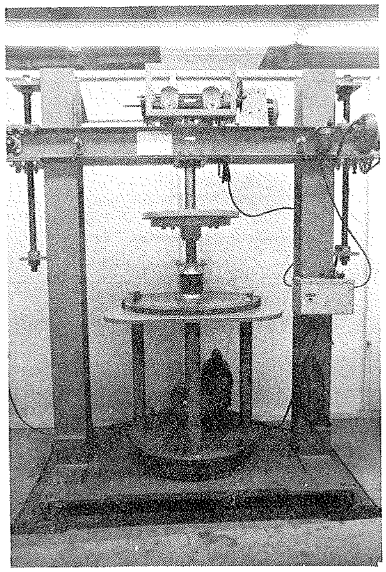


Figure 3. Repeated pore water stressing.

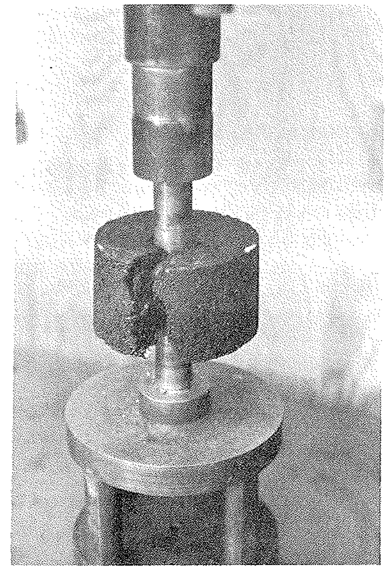


Figure 4. Double punch test.

Table 3. Gradation and SE values of aggregate combinations.

| Item | Percent Passing | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Tucson | | Limestone | | Holbrook No. 1 | | Holbrook No. 2 | |
| | Wet | Washed | Wet | Washed | Wet | Washed | Wet | Washed |
| Gradation, sieve size | | | | | | | | |
| 3/4 in. | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 |
| 1/2 in. | 99 | 99 | 75 | 74 | 87 | 87 | 85 | 85 |
| 3/8 in. | 93 | 93 | 70 | 69 | 73 | 73 | 75 | 74 |
| No. 4 | 64 | 62 | 59 | 57 | 56 | 56 | 48 | 46 |
| No. 8 | 44 | 40 | 45 | 43 | 53 | 52 | 43 | 41 |
| No. 16 | 35 | 31 | 32 | 29 | 45 | 44 | 32 | 30 |
| No. 30 | 26 | 21 | 22 | 19 | 31 | 30 | 28 | 26 |
| No. 50 | 17 | 12 | 15 | 11 | 13 | 12 | 14 | 11 |
| No. 100 | 11 | 5 | 11 | 7 | 3 | 2 | 6 | 3 |
| No. 200 | 7 | 1 | 5 | 1 | 1 | 0 | 4 | 1 |
| Sand equivalent | 31 | 91 | 38 | 85 | 59 | 89 | – | – |

Table 4. Component analysis and consistency of original and reconstituted asphalts.

| Asphalt | Percent of Component | | | | | CRR[a] | Viscosity at 140 F (poise) | Penetration Grade at 77 F |
|---|---|---|---|---|---|---|---|---|
| | A | N | $A_1$ | $A_2$ | P | | | |
| RH | 15.81 | 42.56 | 13.22 | 19.81 | 8.59 | 1.96 | 2,829 | 45 |
| RO | 18.30 | 33.20 | 16.38 | 21.89 | 10.23 | 1.54 | 1,908 | 67 |
| Original | 16.25 | 29.79 | 20.79 | 20.68 | 12.51 | 1.52 | 1,895 | 65 |
| RL | 23.75[b] | 26.43 | 12.33 | 22.93 | 14.57 | 1.03 | 803 | 122 |

Note: 1°C = 5/9 (°F - 32). 1 poise = 0.1 Pa·S.

[a]This is expressed as $(N + A_1)/(P + A_2)$. [b]Extra asphaltene was added to increase viscosity.

equivalent (SE) value than did the Holbrook aggregate, which has a record of poor performance. However, the SE value by itself cannot be used to predict performance, specifically, the stripping of asphaltic concrete.

### Asphalt

The asphalt used in most of this study had a penetration grade of 60-70, which meets specification 705(C) of the Arizona Highway Department (10).

A portion of the study was concerned with the effect of an asphalt's chemical reactivity ratio (CRR) on a paving mixture's resistance to stripping. The CRR is a ratio of sums of components obtained from the Rostler-White (11) component analysis. The composition of the reconstituted asphalts and values of CRR are given in Table 4.

## RESULTS OF VARIOUS TESTING PROGRAMS

The new test developed to detect the stripping susceptibility of asphaltic concrete was investigated for its response to mixture and specimen variations such as asphalt content, type of aggregate, aggregate cleanliness, asphalt composition, and specimen exposures. All specimens were prepared according to a standard procedure. Before mixing, aggregates were heated to 300 ± 10 F (149 ± 6 C); the asphalt was heated to 285 ± 5 F (141 ± 3 C). All compaction was done at an initial mixture temperature of 250 ± 2 F (121 ± 1 C). For the double punch specimen the mixture was compacted with the California kneading compactor by following the general procedure of the Arizona Highway Department (12). The specimens evaluated by the immersion compression method were compacted and tested according to AASHO T167-60 and T165-55 (13).

### Tucson Aggregate

Dry Storage Time—Because the mixture was not aged before compacting or testing, it was necessary to investigate the effects of variable storage or shelf time on the dry and wet double punch strength of an asphaltic mixture. The basic Tucson aggregate (SE 31) with asphalt contents (AC) of 5.5 and 6.0 percent was compacted and the specimens stored in air at 77 F (25.0 C) from 2 to 84 days. The results of these tests are given in Table 5; wet specimens were saturated and stressed for 5,800 repetitions at 122 F (50.0 C) before they were tested at 77 F (25.0 C). A presentation also is shown in Figure 5. The discussion of the results will be based only on strength values.

The curves of Figure 5 indicated that within the range-of-time variable storage time had no great effect on dry strength. However, it had an effect on wet strength; it increased to an optimum value at 7 days and then it decreased. The maximum strength at 7 days might have been caused by the lower void content, but, because saturation was not much different from the value at 84 days, some other factor must have made a greater contribution to the loss in wet strength. Aging of the asphalt and changes in its resistance to the repeated pore water pressure might have been primarily responsible for the change in strength. But, regardless of the actual cause, dry storage time must be kept constant in any comparative strength measurements.

Variable Saturation—Degree of specimen saturation should not affect the tensile strength of asphaltic concrete and, therefore, this variable should not have to be investigated. But, because the variable saturation is obtained after pore pressure stressing, there might be a healing effect caused by drying down from full saturation. The Tucson aggregate specimens were saturated and subjected to pore water pressure stressing. To minimize the effects of storage from stressing to testing, we kept the specimens for high saturation in a 77 F (25.0 C) water bath for 3 days; the specimens for medium saturation were allowed to dry for 3 days in a desiccator at 77 F (25.0 C); the specimens for low saturation were subjected to a vacuum of 10 in. of mercury ($33.8 \times 10^3$ N/m$^2$) on 1 flat surface with laboratory air flowing through parallel to the geometric axis for 10 min. Then the specimens were stored for 3 days in a desiccator at 77 F (25.0 C).

The results of this testing are given in Table 6, and a plot is shown in Figure 6. The relationship between degree of saturation and tensile strength after exposure was some-

Table 5. Effect of variable storage time on tensile strength of Tucson aggregate.

| Time | Density (pcf) | Voids (percent) | Saturation (percent) | Failure Stress (psi) Wet | Failure Stress (psi) Dry | Retained Strength (percent) |
|---|---|---|---|---|---|---|
| 2 days | | | | | | |
| AC 5.0 percent | 141.0 | 6.7 | 128 | 65 | 145 | 45 |
| AC 6.0 percent | 143.0 | 3.8 | 114 | 118 | 149 | 79 |
| 7 days | | | | | | |
| AC 5.0 percent | 144.0 | 4.9 | 118 | 125 | 165 | 76 |
| AC 6.0 percent | 145.0 | 2.6 | 121 | 145 | 164 | 88 |
| 42 days | | | | | | |
| AC 5.0 percent | 140.0 | 7.5 | 128 | 55 | 143 | 38 |
| AC 6.0 percent | 143.5 | 3.7 | 125 | 98 | 162 | 61 |
| 84 days | | | | | | |
| AC 5.0 percent | 139.5 | 7.9 | 119 | 48 | 146 | 33 |
| AC 6.0 percent | 142.5 | 4.8 | 114 | 82 | 141 | 58 |

Note: 1 pcf = 16.0 kg/m$^3$. 1 psi = 6.89 x 10$^3$ N/m$^2$.

Figure 5. Effect of variable storage time on tensile strength of Tucson aggregate.

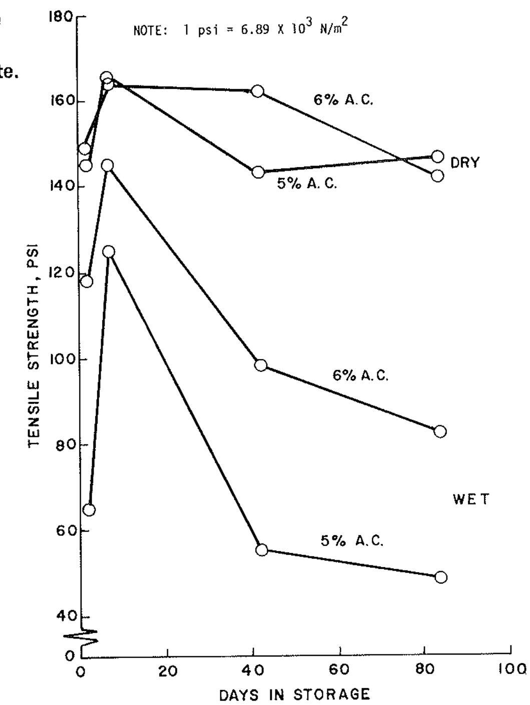



Table 6. Effect of variable saturation on tensile strength after pore water stressing of Tucson aggregate.

| Level of Saturation | Density (pcf) | Voids (percent) | Saturation (percent) | Wet Tensile Strength (psi) |
|---|---|---|---|---|
| Low | | | | |
| AC 5.0 percent | 138.5 | 8.7 | 66 | 22 |
| AC 5.5 percent | 139.0 | 7.4 | 64 | 35 |
| Medium | | | | |
| AC 5.0 percent | 138.0 | 9.0 | 73 | 23 |
| AC 5.5 percent | 139.0 | 7.5 | 91 | 27 |
| High | | | | |
| AC 5.0 percent | 138.5 | 8.6 | 160 | 16 |
| AC 5.5 percent | 139.0 | 7.7 | 158 | 28 |
| Dry | | | | |
| AC 5.0 percent | 138.5 | 8.5 | 0 | 92 |
| AC 5.5 percent | 139.5 | 7.3 | 0 | 119 |

Note: 1 pcf = 16.0 kg/m$^3$. 1 psi = 6.89 x 10$^3$ N/m$^2$

what linear with strength decreasing with increases in degree of saturation. Perhaps of greater interest is the finding that the degree of saturation exceeded 100 percent. This is attributed to the increase of permeable voids in the mineral aggregate from application of the 5,800 repetitions of the fluctuating pore water pressure and to an increase in gross volume and a satisfying aggregate absorption after asphalt film rupture. The amount of water in the specimen divided by the original air void content yielded the degree of saturation. The 2 important findings of this phase were that the void space, through the exposure condition of the test, could be increased and that the period of time from taking the specimen out of the water bath, blotting, weighing, and testing was not a critical one.

Pore Water Pressure Repetitions—The effect of the number of stress repetitions on the wet strength of the Tucson aggregate was established by using 2 asphalt contents and a saturation and stressing temperature of 77 F (25.0 C). The data given in Table 7 show a strength decrease with a loading time increase. The plot shown in Figure 7 shows the strength-number relationship to be linear. Saturation, stressing, and testing took place at 77 F (25.0 C). Future work should include the temperature of 122 F (50.0 C).

Variable Specimen Diameter and Height—Mechanical properties of laboratory prepared specimens are different from those of the same mixture in service. In addition, samples taken from in-service pavements are not of the same dimensions as laboratory-prepared specimens and this difference yields added effects in comparing the properties of specimens. This work attempted to establish geometrical effects on the new method of evaluation for stripping susceptibility. The variables were asphalt content, specimen height, and specimen diameter; because of unforeseen circumstances, the storage time of compacted specimens was an uncontrolled variable. The data for this testing are given in Table 8. The specimens were formed by vibratory kneading compaction. Wet specimens were saturated at 122 F (50.0 C) before they were tested at 77 F (25.0 C). The most acceptable comparison that could be made was that of retained strengths between the specimens 4 and 3.15 in. (80.2 and 101.6 mm) in diameter and it is shown in Figure 8. The retained strengths for specimens of these 2 diameters were essentially the same.

Mixture Variables—The effects of asphalt content and aggregate cleanliness as shown by the SE value on the retained strength according to the new method were determined for the Tucson aggregate. These same variables were used for evaluation by the immersion compression test.

Table 9 gives the results of this testing with AC ranging from 5.0 to 6.0 percent and SE values varying from 31 to 91. The intermediate SE value of 55 was obtained by blending the wet and washed Tucson aggregates having SE values of 31 and 91 respectively. In analyzing the data, one must recognize that the gradation of the 3 blends was not constant. In the double punch method, wet specimens were stressed for 5,800 repetitions at 122 F (50.0 C) before they were tested at 77 F (25.0 C).

A review of the data given in Table 9 shows that the double punch specimens formed by the California kneading compactor had higher densities than those compacted according to the immersion compression method. In general, as the AC and SE values increased, the retained strength increased for both methods of testing.

From the data of Table 9, the degree of saturation of specimens subjected to the repeated pore pressure stressing can be greater than 100 percent.

In Figure 9, the curves show the effects of AC and SE values on the wet and dry strengths obtained by the new method. Generally, the dry strength decreased as the SE value increased. This behavior could be due to the loss of filler material in the blend. Specimen density also decreased with increases of SE value. The same figure shows that the wet strength increased with SE value.

The data plotted in Figure 10 show that according to the new method the retained tensile strength increased as the SE value increased. The point established by a SE value of 91 for the mixture with an AC of 6.0 percent appears to be out of line; however, one must recall that for all 3 values of SE there was a slight variation in filler content.

Figure 11 shows a graphical comparison for retained strengths obtained by the new method and the immersion compression procedure. The plot shows that the new method yields higher values of retained strength than those obtained by the immersion com-

Figure 6. Effect of variable saturation on tensile strength after pore water stressing of Tucson aggregate.

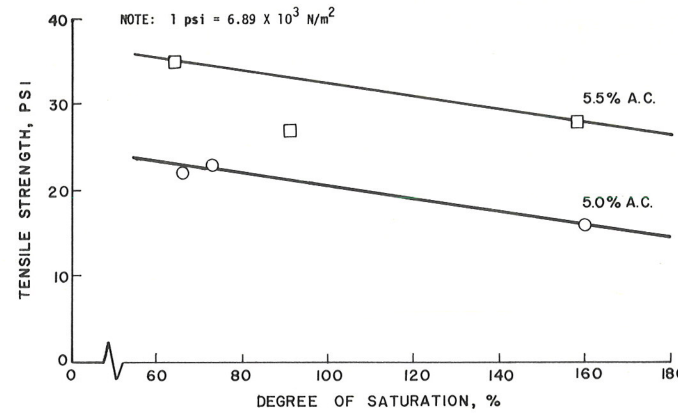



Table 7. Effect of pore water pressure repetitions on tensile strength of Tucson aggregate.

| Stress Applications | Density (pcf) | Voids (percent) | Saturation (percent) | Wet Tensile Strength (psi) |
|---|---|---|---|---|
| 0 | | | | |
| AC 5.0 percent | 141.0 | 7.0 | 95 | 78 |
| AC 5.5 percent | 142.0 | 5.8 | 93 | 106 |
| 5,800 | | | | |
| AC 5.0 percent | 141.0 | 7.0 | 95 | 68 |
| AC 5.5 percent | 142.0 | 5.8 | 93 | 104 |
| 13,200 | | | | |
| AC 5.0 percent | 141.0 | 7.0 | 98 | 55 |
| AC 5.5 percent | 142.0 | 5.8 | 95 | 90 |

Note: 1 pcf = 16.0 kg/m$^3$. 1 psi = 6.89 x 10$^3$ N/m$^2$.

Figure 7. Effect of stress duration on tensile strength of Tucson aggregate.

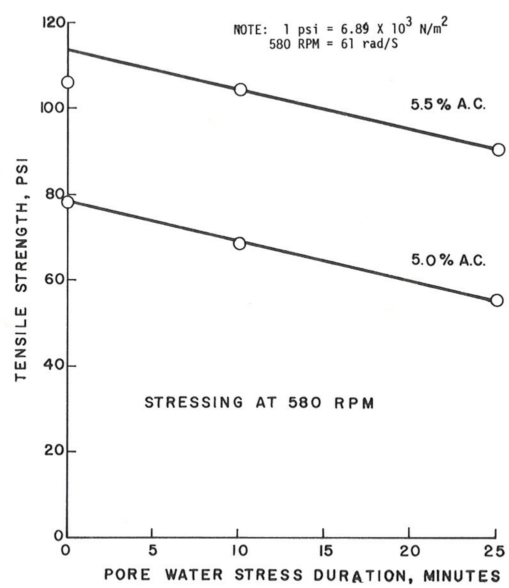


**Table 8. Effect of variable specimen diameter and height on tensile strength of Tucson aggregate.**

| Specimen Size (in.) | Density (pcf) | Voids (percent) | Failure Stress (psi) Wet | Dry | Retained Strength (percent) |
|---|---|---|---|---|---|
| 3.15 by 2 | | | | | |
| AC 5.0 percent | 142.0 | 6.2 | 164 | 322 | 51 |
| AC 5.5 percent | 144.0 | 4.4 | 134 | 264 | 51 |
| AC 6.0 percent | 144.5 | 3.0 | 211 | 274 | 77 |
| 3.15 by 4 | | | | | |
| AC 5.0 percent | 143.0 | 5.5 | 89 | 161 | 55 |
| AC 5.5 percent | 144.5 | 3.8 | 159 | 196 | 81 |
| AC 6.0 percent | 146.5 | 1.6 | 130 | 132 | 99 |
| 4.0 by 2 | | | | | |
| AC 5.0 percent | 143.5 | 5.1 | 137 | 219 | 63 |
| AC 5.5 percent | 144.5 | 3.9 | 105 | 201 | 52 |
| AC 6.0 percent | 145.5 | 2.3 | 200 | 250 | 80 |
| 4.0 by 4 | | | | | |
| AC 5.0 percent | 142.0 | 6.1 | 70 | 148 | 47 |
| AC 5.5 percent | 147.5 | 1.9 | 136 | 160 | 85 |
| AC 6.0 percent | 144.5 | 2.9 | 77 | 142 | 54 |

Note: 1 pcf = 16.0 kg/m$^3$. 1 psi = 6.89 x 10$^3$ N/m$^2$.

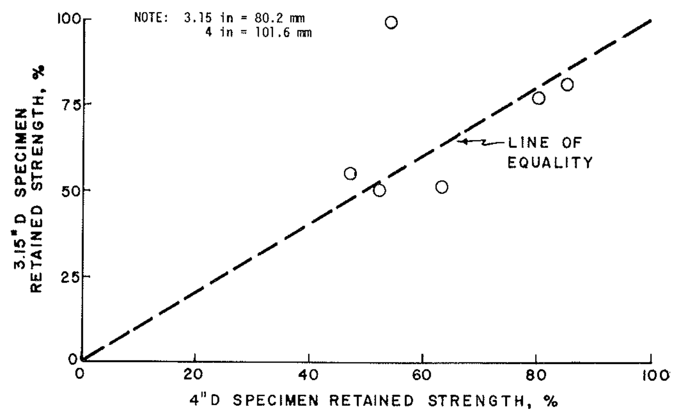



**Figure 8. Relationship between the retained strength of 4-in.- and 3.15-in.-diameter specimens with variable AC.**

**Table 9. Effect of AC and SE value on wet and dry strengths of Tucson aggregate.**

| Value | Density (pcf) | Voids (percent) | Saturation (percent) | Failure Stress (psi) Wet | Dry | Retained Strength (percent) |
|---|---|---|---|---|---|---|
| Double Punch Method | | | | | | |
| Sand equivalent 31 | | | | | | |
| AC 5.0 percent | 141.0 | 6.8 | 107 | 43 | 124 | 35 |
| AC 5.5 percent | 141.5 | 5.7 | 128 | 58 | 118 | 49 |
| AC 6.0 percent | 144.0 | 3.6 | 116 | 76 | 104 | 73 |
| Sand equivalent 55 | | | | | | |
| AC 5.0 percent | 140.0 | 7.6 | 116 | 61 | 113 | 54 |
| AC 5.5 percent | 141.0 | 6.1 | 110 | 83 | 105 | 79 |
| AC 6.0 percent | 144.0 | 3.7 | 117 | 123 | 135 | 91 |
| Sand equivalent 91 | | | | | | |
| AC 5.0 percent | 140.5 | 7.2 | 106 | 72 | 85 | 85 |
| AC 5.5 percent | 139.5 | 7.2 | 105 | 73 | 79 | 92 |
| AC 6.0 percent | 142.0 | 4.8 | 99 | 85 | 110 | 77 |
| Immersion Compression Method | | | | | | |
| Sand equivalent 31 | | | | | | |
| AC 5.0 percent | 137.5 | 9.2 | 99 | 170 | 553 | 31 |
| AC 5.5 percent | 138.5 | 7.9 | 88 | 192 | 421 | 46 |
| AC 6.0 percent | 139.5 | 6.5 | 72 | 275 | 502 | 55 |
| Sand equivalent 55 | | | | | | |
| AC 5.0 percent | 135.0 | 10.4 | 102 | 159 | 346 | 46 |
| AC 5.5 percent | 136.0 | 9.1 | 107 | 217 | 346 | 63 |
| AC 6.0 percent | 137.0 | 7.1 | 86 | 257 | 406 | 63 |
| Sand equivalent 91 | | | | | | |
| AC 5.0 percent | 135.5 | 10.5 | 92 | 162 | 307 | 53 |
| AC 5.5 percent | 134.5 | 10.5 | 92 | 200 | 341 | 59 |
| AC 6.0 percent | 135.5 | 9.2 | 74 | 282 | 438 | 64 |

Note: 1 pcf = 16.0 kg/m$^3$. 1 psi = 6.89 x 10$^3$ N/m$^2$.

Figure 9. Relationship between SE value and tensile strength as affected by AC for the Tucson aggregate.

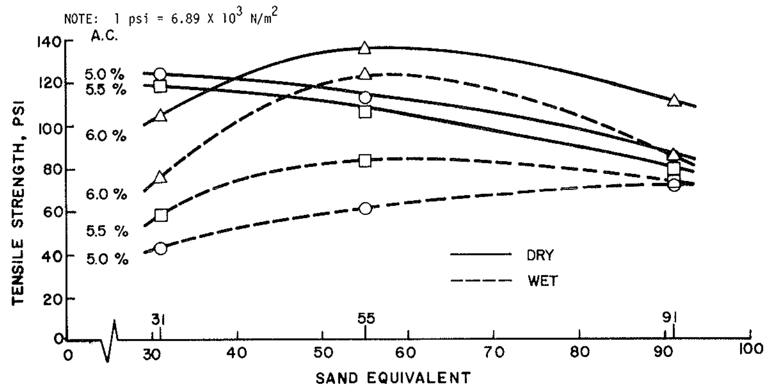



Figure 10. Relationship between SE value and retained tensile strength for the Tucson aggregate.

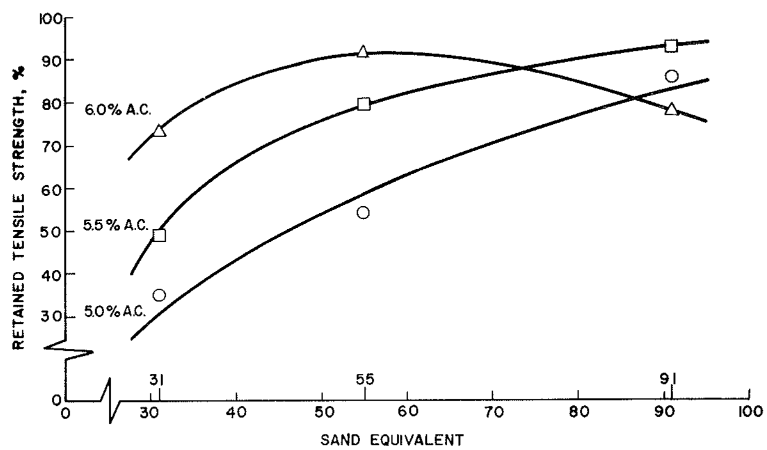
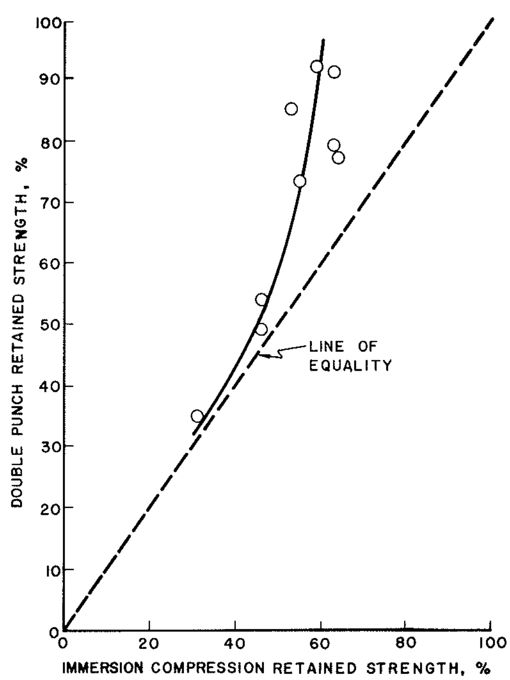



Figure 11. Relationship between retained strengths obtained by the double punch method and the immersion compression method for the Tucson aggregate.



pression procedure. Because the slope of the curve is greater than 1, the new procedure was more sensitive to the variables than was the immersion compression method. Higher value does not mean better value.

### Limestone Aggregate

The evaluation of this blend included SE value and testing method variables. Table 10 gives the specimen characteristics for mixtures evaluated by the new and immersion compression methods. In the double punch method wet specimens were stressed for 5,800 repetitions at 122 F (50.0 C) before they were tested at 77 F (25.0 C). The values of particular interest are for the degree of saturation and retained strength. It is apparent that the exposure conditions of both methods resulted in significant saturation of the specimens. The aggregate blend had been presented as having a good performance record. The data of Table 10 indicate that the new method, for SE values of 38 and 85, yielded retained strengths of 66 and 78 percent respectively. The immersion compression test yielded retained strengths of 36 and 53 percent respectively.

### Holbrook Aggregate (No. 1)

Because it was known that the complete aggregate blend as it came from the pit contained some clay, an elaborate procedure of separating, soaking, elutriating, boiling, and elutriating again was used to cleanse the aggregate of the deleterious clay. Table 11 gives specimen characteristics for the stripping resistance of asphaltic concrete made with this aggregate. In the double punch method, wet specimens were stressed for 5,800 repetitions at 122 F (50.0 C) before they were tested at 77 F (25.0 C). Of particular interest are the relatively high values for degree of saturation for both types of specimen; it appears that aggregate characteristics rather than exposure conditions are responsible for the degree of saturation for the immersion compression and double punch specimens.

The change in SE value from 59 to 89 did not cause a significant change in retained strength according to either method. Because an appreciable change in gradation was not caused by the cleansing of the aggregate, the poor resistance to stripping seemed to be due to aggregate surface texture and composition of the aggregate.

### Asphalt Composition

Asphalt composition may contribute to the stripping susceptibility of an asphaltic mixture. To investigate this, we decided to separate the basic asphalt into its Rostler-White components and then recombine it to obtain a range of CRR values from low to high. These reconstituted asphalts were then used with a second Holbrook aggregate blend. These materials have been described earlier in Tables 3 and 4. Because the main effects were to be related to asphalt composition, voids and saturation measurements were not made for the specimens tested by the new procedure.

The effects of asphalt composition on the wet and dry strength obtained by the new procedure can be seen from the data given in Table 12. In the double punch method, wet specimens were stressed for 5,800 repetitions at 122 F (50.0 C) before they were tested at 77 F (25.0 C). The compositional differences of the asphalts are presented by CRR. These data indicate that the CRR did affect the wet and dry strength separately but there was no significant difference in retained strength for the various CRRs. A graphical representation of the results for a blend with an AC of 5.5 percent is shown in Figure 12. Generally, the wet and dry strengths increased as CRR and viscosity at 140 F (60.0 C) increased.

## DISCUSSION AND CONCLUSIONS

### Testing Procedure

The general testing procedure appeared to be adequate for laboratory evaluation of asphaltic mixtures. The conditions set for saturation, stressing, and strength testing were reasonable for time, temperature, and repeatability. Of particular interest are

**Table 10. Effect of SE value on wet and dry strengths of limestone aggregate.**

| Item | Sand Equivalent Values[a] 38 | 85 |
|---|---|---|
| Double Punch Method | | |
| Density, pcf | 141.5 | 139.0 |
| Voids, percent | 4.0 | 5.8 |
| Saturation, percent | 158 | 142 |
| Failure stress, psi | | |
| Wet | 96 | 107 |
| Dry | 144 | 137 |
| Retained strength, percent | 66 | 78 |
| Immersion Compression Method | | |
| Density, pcf | 139.5 | 137.0 |
| Voids, percent | 6.6 | 8.1 |
| Saturation, percent | 135 | 133 |
| Failure stress, psi | | |
| Wet | 222 | 199 |
| Dry | 620 | 379 |
| Retained strength, percent | 36 | 53 |

Note: 1 pcf = 16.0 kg/m$^3$. 1 psi = 6.89 x 10$^3$ N/m$^2$.
[a]Asphalt content was 5.5 percent.

**Table 11. Effect of SE value on wet and dry strengths of Holbrook aggregate No. 1.**

| Item | Sand Equivalent Values[a] 59 | 89 |
|---|---|---|
| Double Punch Method | | |
| Density, pcf | 144.0 | 142.0 |
| Voids, percent | 2.3 | 4.0 |
| Saturation, percent | 281 | 212 |
| Failure stress, psi | | |
| Wet | 59 | 54 |
| Dry | 89 | 95 |
| Retained strength, percent | 65 | 57 |
| Immersion Compression Method | | |
| Density, pcf | 138.5 | 136.5 |
| Voids, percent | 6.5 | 8.5 |
| Saturation, percent | 154 | 144 |
| Failure stress, psi | | |
| Wet | 110 | 94 |
| Dry | 274 | 161 |
| Retained strength, percent | 40 | 58 |

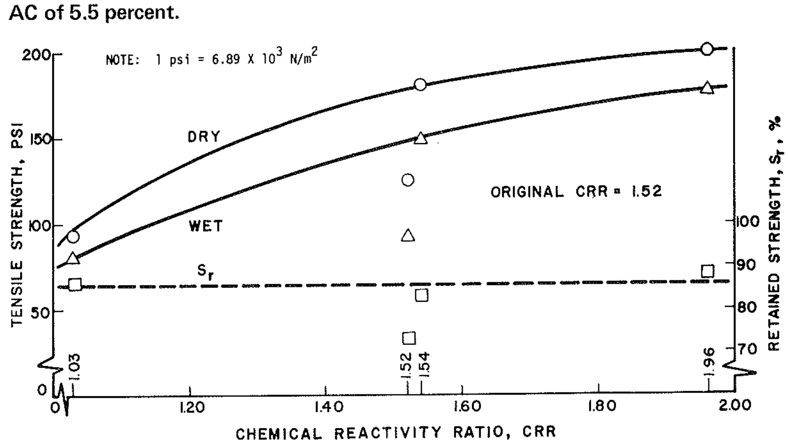
Note: 1 pcf = 16.0 kg/m$^3$. 1 psi = 6.89 x 10$^3$ N/m$^2$.
[a]Asphalt content was 5.5 percent.

**Table 12. Effect of CRR on wet and dry strengths of Holbrook aggregate No. 2.**

| Value | Density (pcf) | Failure Stress (psi) Wet | Dry | Retained Strength (percent) |
|---|---|---|---|---|
| CRR 1.52[a] | | | | |
| AC 5.0 percent | 148.0 | 97 | 143 | 68 |
| AC 5.5 percent | 147.0 | 92 | 125 | 73 |
| AC 6.0 percent | 147.0 | 99 | 109 | 91 |
| CRR 1.54[b] | | | | |
| AC 5.0 percent | 147.5 | 143 | 180 | 79 |
| AC 5.5 percent | 146.5 | 148 | 180 | 83 |
| AC 6.0 percent | 147.0 | 138 | 173 | 80 |
| CRR 1.96[b] | | | | |
| AC 5.0 percent | 146.0 | 133 | 190 | 70 |
| AC 5.5 percent | 147.0 | 176 | 199 | 88 |
| AC 6.0 percent | 147.0 | 193 | 199 | 97 |
| CRR 1.03[b] | | | | |
| AC 5.0 percent | 146.0 | 62 | 83 | 75 |
| AC 5.5 percent | 147.5 | 80 | 93 | 86 |
| AC 6.0 percent | 147.0 | 89 | 85 | 107 |

Note: 1 pcf = 16.0 kg/m$^3$. 1 psi = 6.89 x 10$^3$ N/m$^2$.
[a]Original. [b]Reconstituted.

**Figure 12. Effect of CRR on tensile strength of Holbrook aggregate No. 2 with an AC of 5.5 percent.**



the measurements for degree of saturation and the method for applying the repetitive loads for the exposure of the wet specimens.

There appeared to be some relationship between degree of saturation of different aggregate mixtures and their resistance to stripping. It was difficult to accept that in-service paving mixtures dilate under the action of water or pore water pressure; nevertheless, it was apparent that water-caused permeable volume changes would indicate a water-susceptible mixture.

The repetitive pore water pressure given the specimens was effected through a unique device. The device may not have been important but the rate of loading was in that pore pressure surging and sand particle erosion were affected by cycling rate.

The double punch test was found to be simple and repeatable; however, it suffered, as did most others, from effects on strength caused by specimen geometry. As such the test will require that specimen size be standardized, even though the data presented showed that retained strength may not have been affected by specimen geometry.

Comparison between the double punch and immersion compression wet and dry strengths only indicated that the 2 tests rated the mixtures in essentially the same order. In general, the new method yielded higher values of retained strength.

### Material Variables

The data obtained with the new procedure indicated that the resistance to stripping was improved with increases in AC and SE values. But this was expected based on prior experience.

Selection of the limestone and Holbrook aggregates for the study was based on good performance for the limestone and poor for the Holbrook. The service record for the Tucson aggregate was considered to have been acceptable for the region. For the natural aggregates 5.5 percent AC seemed to have been the design amount for the limestone and Tucson aggregates, but it was 0.5 percent high for the Holbrook No. 1 aggregate. For these conditions, the double punch test rated the limestone mixture as the best and the Tucson aggregate as the worst; the immersion compression test rated the Tucson aggregate as the best and the limestone as the worst for retained strength. The terms best and worst are relative and are not meant to be related to acceptable and rejectable.

The effect of CRR on retained strength according to the new method was surprising. The data showed that the higher the CRR value, the greater the viscosity. Our general experience has been that higher retained strengths are obtained with the immersion compression method for mixtures having asphalt with higher viscosity. In this case, retained strength according to the new method was independent of viscosity or CRR value.

An interesting finding in the asphalt composition study was the greatly improved value of retained strength obtained for the Holbrook aggregate. The gradation for Holbrook No. 2 was improved, and greater amounts of asphalt were used to obtain the improved values of wet, dry, and retained strength.

## CONCLUSIONS

Careful examination of the data together with experience in evaluating durability of asphaltic concrete leads to the following conclusions:

1. The new procedure for evaluating the stripping susceptibility of asphaltic concrete is simple and repeatable;
2. The responses to test and material variables generally follow established trends;
3. At present, one must fix specimen and punch sizes for comparison of different mixtures to eliminate geometrical effects; and
4. As with any new mixture evaluation procedure, the new procedure must be field tested.

The new procedure to evaluate the debonding susceptibility of asphaltic concrete was based on the following:

1. The way to determine the presence of debonding is with a tension test;
2. The action that causes debonding in the field should be simulated in the laboratory; and
3. The procedure should be capable of testing a pavement core without remolding.

Because these considerations are incorporated in the new procedure and because the work presented has indicated acceptable responses for material variables, repeatability, and simplicity, we recommend that the debonding test procedure be used to evaluate standard Hveem specimens prepared for the routine testing of asphaltic concrete by the Arizona Highway Department. To aid in establishing required minimum strength and/or retained strength, one should sample in-service mixtures on a continuing basis to compare with results obtained from laboratory-prepared specimens.

## ACKNOWLEDGMENT

I would like to recognize the contributions of personnel from the Materials and Test Section of the Arizona Highway Department, who obtained the aggregates, and also those who separated and recombined the asphalt for the CRR variation study. Don Stout and Elmer Green were particularly helpful. The contents of this report reflect the views of the author, who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Arizona Highway Department or the Federal Highway Administration.

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