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TESTING FOR DEBONDING of ASPHALT FROM AGGREGATES

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Final Report

TESTING FOR DEBONDING OF ASPHALT FROM AGGREGATES

by

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Submitted to

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Phoenix, Arizona 85007

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16. Abstract

The report 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"D X 2 1/2"H) was water saturated at 122°F. Then the specimen was subjected to repeated pore water pressure. The effects 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.

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Final Report

TESTING FOR DEBONDING OF ASPHALT FROM AGGREGATES

INTRODUCTION

The topic of asphalt stripping in pavements has been of interest for many years. The phenomenon of stripping has resulted in complete failure of a pavement within two weeks after opening to traffic to failure by loss of surface fines over a period of two years. In order to minimize the expense and inconvenience resulting from rebuilding and resurfacing of a damaged surface, much effort has been and is being expended to define, measure and predict the occurrence of asphalt stripping.

The objective of the investigation was to develop an improved method for determining the susceptibility to stripping of asphaltic concrete mixtures. The discussion to follow covers a brief review of various methods that have been used and also the new procedure and results of tests performed on various mixtures.

REVIEW

Factors Contributing to Stripping

Early failure of asphalt pavement surfaces has generally been attributed to stripping since cracking due to structural failure should not be expected in the very early life of a highway, and all stripping failures have been associated with the presence of water (1)*. The term "stripping" or "debonding" will be used in the text since these are presently used in the jargon of asphalt technology. Stripping and debonding imply a surface-to-surface separation; however, Schmidt (2) and others suggest that these are not necessarily adhesion failures but may be attributed to cohesion deficiency. It is not our intent to define or describe the mechanism(s) of the stripping failure, but we can list the factors which have been reported to contribute to the stripping occurrence. These are enumerated as follows:

1. Water is always involved.
2. Traffic is always a factor.
3. Cool temperatures.
4. Low asphalt content.
5. Low asphalt viscosity.
6. High air void content.
7. Cleanliness of fine aggregate.

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* The numbers in parentheses correspond to the references listed at the end of the report.

8. Coating on aggregate.
9. Composition of aggregate.
10. Surface texture of aggregate.

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

Test Procedures

Unconfined Compression

The best known procedure for determining the susceptibility to stripping of asphaltic concrete is the one known as the immersion compression test; the ASTM designation is D 1075-54 and AASHO's is T165-55. This procedure has been used extensively since a numerical index for the complete and compacted design mixture is obtained for a measure of the loss of cohesion resulting from the action of water. Specimens are compacted by double-plunger action at a pressure of 3000 psi. The exposed specimens are generally soaked for 24 hours in a water bath held at 140^{o}F. Testing is performed under unconfined compression at a deformation rate of 0.20 in./min. for a 4-inch high specimen and at a temperature of 77^{o}F.

The index of retained strength is obtained by dividing the strength of the exposed specimens to 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 the resistance to stripping in much the same manner as the immersion compression test. Research done by the author (4) has shown that for certain aggregate-asphalt mixtures the results obtained by this procedure can be quite misleading.

Confined Compression

The loss of strength or of stiffness of asphaltic concrete due to the action of water and repeated stressing has been determined by Majidzadeh and Stander (5) and by Terrel (6). In these works, triaxial compression with a repeated deviator stress has been utilized.

Split Cylinder Test

The most recent method for measuring the stripping susceptibility of asphaltic concrete is one based on the split cylinder test and proposed by Lottman (7). A "retained strength" index is obtained from measurements of indirect tensile strength for specimens (or cores) 4 inches in diameter. Exposure was achieved by 12 freeze-thaw cycles of 0^{o}-120^{o}-0^{o}F of water saturated specimens. Strength testing was done at 55^{o}F and with a deformation rate of 0.065 inch per minute.

The reading of the above citations and our own thoughts have suggested that there must be a simpler and, at least just, as effective method for determining the susceptibility to stripping or debonding of asphaltic concrete.

NEW TEST PROCEDURE

The development of the new test procedure came about from visual observation, reading of the literature and prior experience in making measurements of asphalt stripping. We have noticed that loss of cover (surface) 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 to cause raveling. Complete disintegration or cracking of the asphaltic layer because of 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. However, visual examinations and research into these failures indicate that stripping or debonding was brought about with the action(s) of water.

The basic concepts for the new debonding susceptibility test are listed below.

1. Specimen size is to be approximately of standard laboratory size of 4 inches in diameter by 2 1/2 inches high. However, field cores of 3-4 inches in diameter and 2-4 inches in height can be tested.
2. Specimen is to be saturated since pore water pressure is to be used in the exposure portion of the test and it is believed that such pressure develops in the field condition.
3. The exposure of the specimen is to be a repeated pore water

pressure only at 122^{o}F. The use of pore water pressure only is to simplify the loading of different sized specimens. The temperature of 122^{o}F is to approach the value found by the author (8) for saturated pavements in Southern Arizona and to reduce asphalt viscosity and thus resistance to debonding. The use of a 122^{o}F temperature rather than a "cold rain" temperature discussed above was based on the thought that the loss of surface material in raveling was due to a brittle failure rather than an adhesive failure which we relate directly to stripping or debonding.

4. The strength test is to be simple, repeatable, and be of the tension type. The test temperature for strength is to be 77^{o}F.

Development of the concepts into specifics follows.

Strength Test

The effects of stripping or debonding of asphalt from the aggregate of asphaltic concrete should be most detectable by means of a tension test. Since a direct tension test is not practical, some sort of a simple indirect tension test such as the cohesiometer, Marshall, split cylinder, or recent double punch was considered. Prior experiences with the cohesiometer and Marshall were not particularly good and thus were not used. The work of Lottman (7) and others suggested the split cylinder procedure. However, the work reported by Fang and Chen (9) on 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 were carried out with two mixtures of asphaltic concrete compacted by vibratory kneading compaction (10). The split cylinder specimens were loaded through 1-inch wide steel bars and the double punch test utilized 1-inch diameter punches. Table 1 shows the repeatability of the two test methods and the relationship between the stresses obtained for two aggregate gradations. The indications of the data are that the double punch test has better repeatability than the split cylinder and that the stresses obtained by the two procedures were essentially identical to each other since the value of the slope coefficient is almost one.

Observations of these two test methods indicate that the double punch test was simpler in operation and the stress analysis does not have to be corrected for flattening of the load surface as in the split cylinder.

Specimen Size

Another preliminary work with the double punch test was concerned with the size of specimen and also punch diameter. Variations in specimen size covered diameters of 2, 4 and 6 inches and heights of 2, 4, and 8 inches; while punch diameters were 3/8, 5/8 and 1 inch. The results of this testing appear in Table 2. The data indicate the following:

1. For constant specimen height, the tensile stress decreases

Table 1 Tensile Stress by Indirect Methods on Asphaltic Concrete Specimen 4"D X 2 1/2"H

Deformation in/min.	Test Temp. °F	Split Cylinder Stress psi $S_{S.C.}$	Split Cylinder Coeff. of Variation %	Double Punch Stress psi $S_{D.P.}$	Double Punch Coeff. of Variation %
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	---	---

$S_{S.C.} = 1.14\ S_{D.P.} - 12.2$

$r^2 = 0.94$

Deformation in/min.	Test Temp. °F	Split Cylinder Stress psi $S_{S.C.}$	Split Cylinder Coeff. of Variation %	Double Punch Stress psi $S_{D.P.}$	Double Punch Coeff. of Variation %
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	---	---

$S_{S.C.} = 1.07\ S_{D.P.} + 0.8$

$r^2 = 0.98$

Specimens compacted by vibratory kneading and tested in triplicate.

(Reference 15)

Table 2 Effects of Specimen and Double Punch Size on Tensile Strength of Asphaltic Concrete

Specimen Size (D"XH")	Punch Diam., in.	Tensile Strength, psi	Sample Stand. Dev.	Coeff. of Var. %
6 X 2	1	145	11.4	7.9
4 X 2	1	150	5.6	3.7
2 X 2	1	284	22.2	7.9
6 X 2	5/8	82	4.5	5.6
4 X 2	5/8	74	2.1	2.8
2 X 2	5/8	130	15.0	11.6
4 X 2	3/8	47	1.7	3.7
2 X 2	3/8	52	12.5	24.1
4 X 4	5/8	42	0.6	1.4
4 X 8	5/8	22 (Not a tensile failure)		
4 X 8	1	40 (Not a tensile failure)		

Specimens compacted by vibratory kneading, tested in triplicate, at 77°F, and deformation rate of 1 inch per minute.

(Reference 16)

as the ratio of specimen diameter to punch diameter (D/d) increases.

2. For constant specimen height and D/d ratio, the tensile strength decreases as specimen diameter decreases.

Additional data related to specimen size and obtained with the project aggregates will be presented later.

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 to saturate a specimen. Our work was centered about a mixture with relatively good resistance to the action of water and specimens compacted with the California kneading compactor. It was established that a submerged specimen could be fully saturated with distilled water with a vacuum of 20 inches of mercury held for 5 minutes and then allowing 5 more minutes at atmospheric pressure for a "blotting" effect. The temperature of the water was that of the laboratory or about 75^{o}F.

Double punch tensile tests performed on control and vacuum-saturated specimens indicated no adverse effect or loss of strength was caused by the vacuum-saturation procedure. However, later work with a more water susceptible mixture and vacuum-saturation at 122^{o}F, indicated that this combination of exposure can cause a loss of tensile strength by as much as 20 per cent of the dry (control) specimen.

Specimen Stressing

The desire to simulate traffic loading conditions that cause debonding resulted in considering a repeated pore water pressure stressing for specimen exposure. The stressing or conditioning of a specimen follows the vacuum-saturation of the specimen at 122^{o}F. 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 which caused a hydraulic pressure varying from 5 to 30 pounds per square inch at a rate of 580 times per minute. Since the saturation chamber was of clear plastic, the "muddying" of water and surging of loose sand particles could be observed as the stressing of a specimen was continued over a period of time.

It is considered that the repeated pore water pressure and surging of sand particles within the saturated specimen approaches the stressing that a rain soaked pavement receives from moving traffic loads.

Photographs of the equipment and procedure for performing the new debonding test appear in Appendix A.

MATERIALS FOR PROCEDURAL APPRAISAL

In order to obtain information related to responses of the new debonding test to different variables a testing program was developed around Arizona aggregates, a standard aggregate, and a stripping test.

Materials

Aggregates

Three general aggregate blends were selected to represent the best, average, and worst types. The Tucson aggregate was typical of the aggregate used for asphaltic concrete around the Tucson area and was chosen because of its availability and being representative of average aggregates obtained from dry washes and pits.

The Holbrook aggregate was selected because it represented the worst type or most susceptible to debonding. Also the Arizona Highway Department has accumulated a great deal of information concerning the stripping characteristics of asphaltic concrete using the Holbrook aggregate.

The Limestone aggregate was selected as one having a good performance record with reference to resistance to debonding.

Aggregate characteristics are listed in Table B1 of Appendix B. It is noted from this table that there is an inconsistency between sand equivalent value and performance in that the Limestone aggregate has a lower (worse) sand equivalent value than the Holbrook aggregate which has a record of poor performance. However, it is generally recognized that the sand equivalent value by itself can not be used to predict performance with reference to stripping of asphaltic concrete.

Asphalt

The asphalt used throughout the greater portion of the study was of 60/70 penetration grade meeting specifications 705(C) of the Arizona Highway Department (11).

A portion of the study was concerned with the effect of an asphalt's "chemical reactivity ratio" (CRR) on the resistance to stripping of a paving mixture. The CRR is a ratio of sums of components obtained from the Rostler-White (12) component analysis. The composition of the reconstituted asphalts and values of CRR are shown in Table B2 of Appendix B. It had been the intent to have had one of the reconstituted asphalts made to the same proportions as the original, but as can be seen such was not the case.

RESULTS OF VARIOUS TESTING PROGRAMS

The new test developed for detecting the stripping susceptibility of asphaltic concrete was investigated to determine its response to mixture and specimen variations such as asphalt content, aggregate type, aggregate cleanliness, asphalt composition, and specimen exposures. The results and discussions of these tests are presented under the headings of Tucson, Holbrook, Limestone Aggregates, and Asphalt Composition.

All specimens were prepared following a standard procedure. Aggregates were heated to 300 $\pm$ 10^{o}F and the asphalt was heated to 285 $\pm$ 5^{o}F prior to mixing. All compaction was done at an initial mixture temperature of 250 $\pm$ 2^{o}F. For the double punch specimen the mixture was compacted with the California kneading compactor following the general procedure of the Arizona Highway Department (13). The specimen evaluated by the "immersion-compression" method were compacted and tested in general accordance with AASHO T 167-60 and T 165-55 (14).

Tucson Aggregate

Test Variables

Dry Storage Time. Since there was no aging of the mixture prior to compaction nor prior to 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 of 5.5 and also 6.0 percent was compacted and the specimens stored in air at 77^{o}F for periods ranging from 2 to 84 days. The results of these tests appear in Table C1 of Appendix C and a more visual presentation is given in Figure 1. At this time, the discussion of the results will be based only with reference to the strength values.

The curves of Figure 1 indicate that within the range of time variable, storage time had no great effect on dry strength. However the wet strength was affected by storage time in that it increased to an optimum value at 7 days and then decreased. The maximum strength at 7 days might be caused by the lower void content but since saturation is not much different from the value at 84 days then we suspect that some other factor has greater contribution to the loss in wet strength. It is proposed that aging of the asphalt and changes in its resistance to the repeated pore water pressure are primarily responsible for the change in strength. Regardless of the actual cause, it is apparent that the dry storage time must be kept constant in any comparative strength measurements.

Variable Saturation. It would seem that the degree of specimen saturation should not affect the tensile strength of asphaltic concrete and, therefore, there should be no need to investigate this variable. However, since the variable saturation would be obtained after pore pressure stressing, it was believed that there might be a healing effect due to drying down from full saturation. The Tucson aggregate specimens were saturated and pore water pressure stressed by the standard procedure. In order to minimize the effects of storage, from stressing to testing, the specimens for high saturation were kept in a 77^{o}F water bath for 3 days; the specimens for medium saturation were allowed to dry for 3 days in a desicator at 77^{o}F, and those for low saturation were subjected to a vacuum of 10 in. of mercury on one flat surface with laboratory air flowing through

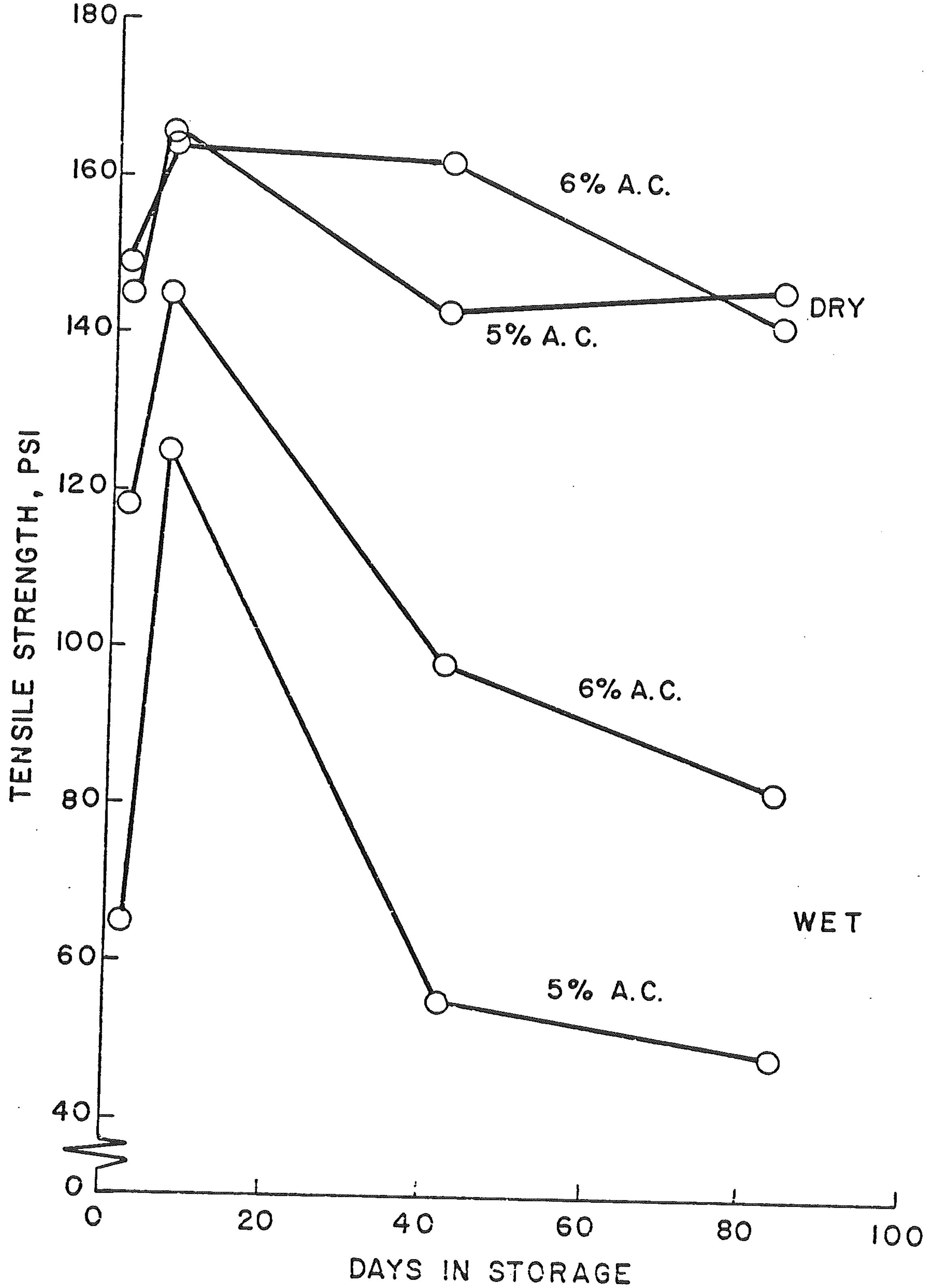

FIGURE 1 Effect of Storage Time on Wet and Dry Tensile Strength of Asphaltic Concrete

parallel to the geometric axis for 10 minutes. Then the specimens were stored for 3 days in a desiccator at 77^{o}F.

The results of this testing are listed in Table C2 and a plot is shown in Figure 2. The relationship between degree of saturation and tensile strength after exposure appears to be somewhat linear with the strength decreasing with increases in degree of saturation. Perhaps of greater interest is the finding that the degree of saturation exceeded 100 percent. The behavior is attributed to the increase of permiable voids in the mineral aggregate due to the application of the 5800 repetitions of the fluctuating pore water pressure. The excess saturation is attributed to increase in gross volume and 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 two important findings of this phase were the possibility of increasing the void space through the exposure condition of the test and also that the time element from taking the speciment out of the water bath, blotting, weighing, and testing is 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 using two asphalt contents and a saturation and stressing temperature of 77^{o}F. The data of Table C3 in Appendix C show a strength decrease as loading time increased. The plot of Figure 3 shows the strength-number relationship to be quite linear. The stressing temperature of 77^{o}F was used because of convenience; however, future work should include the temperature of 122^{o}F.

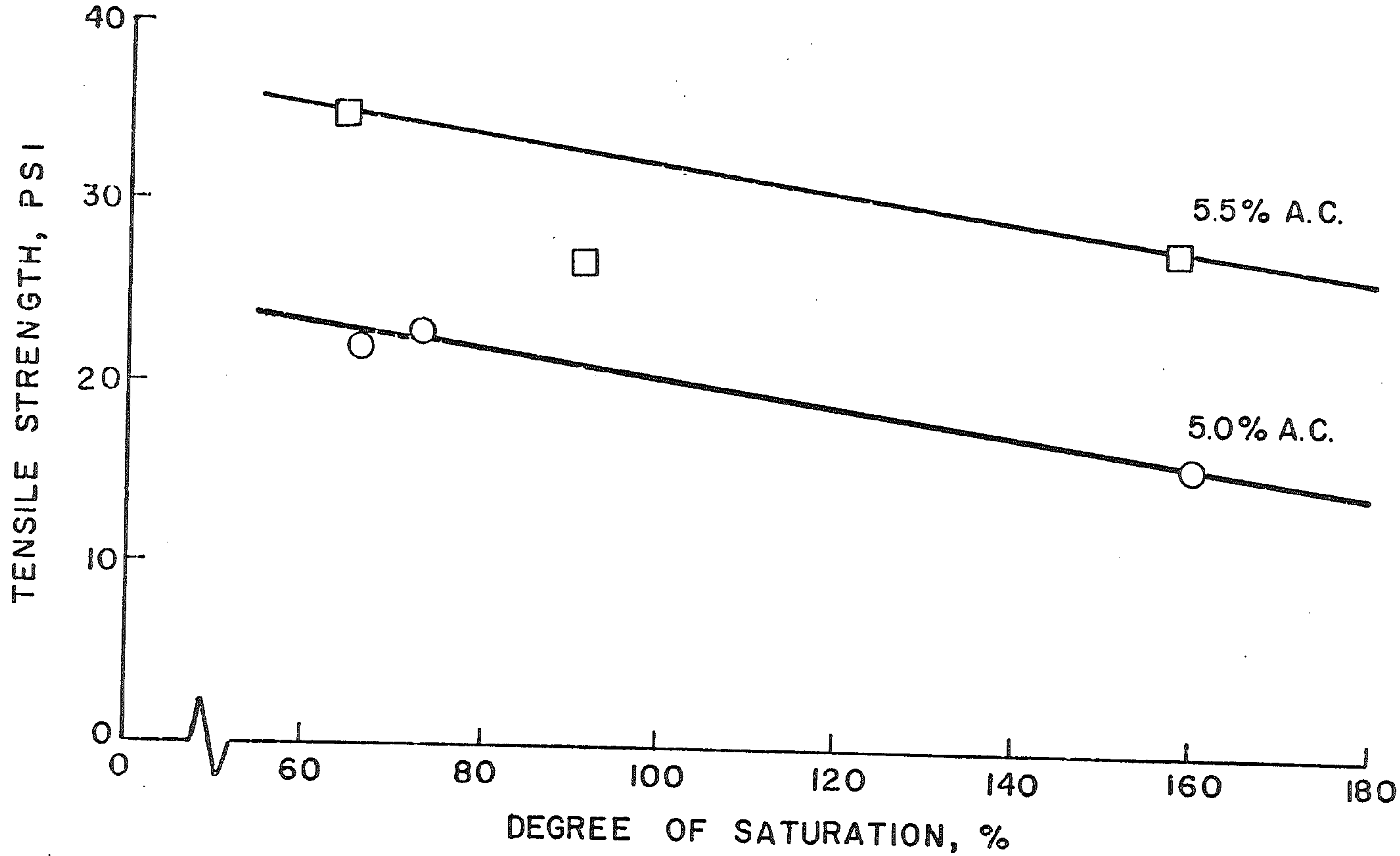

FIGURE 2 Effect of Variable Saturation on Wet Tensile Strength After Pore Water Stressing of Tucson Aggregate

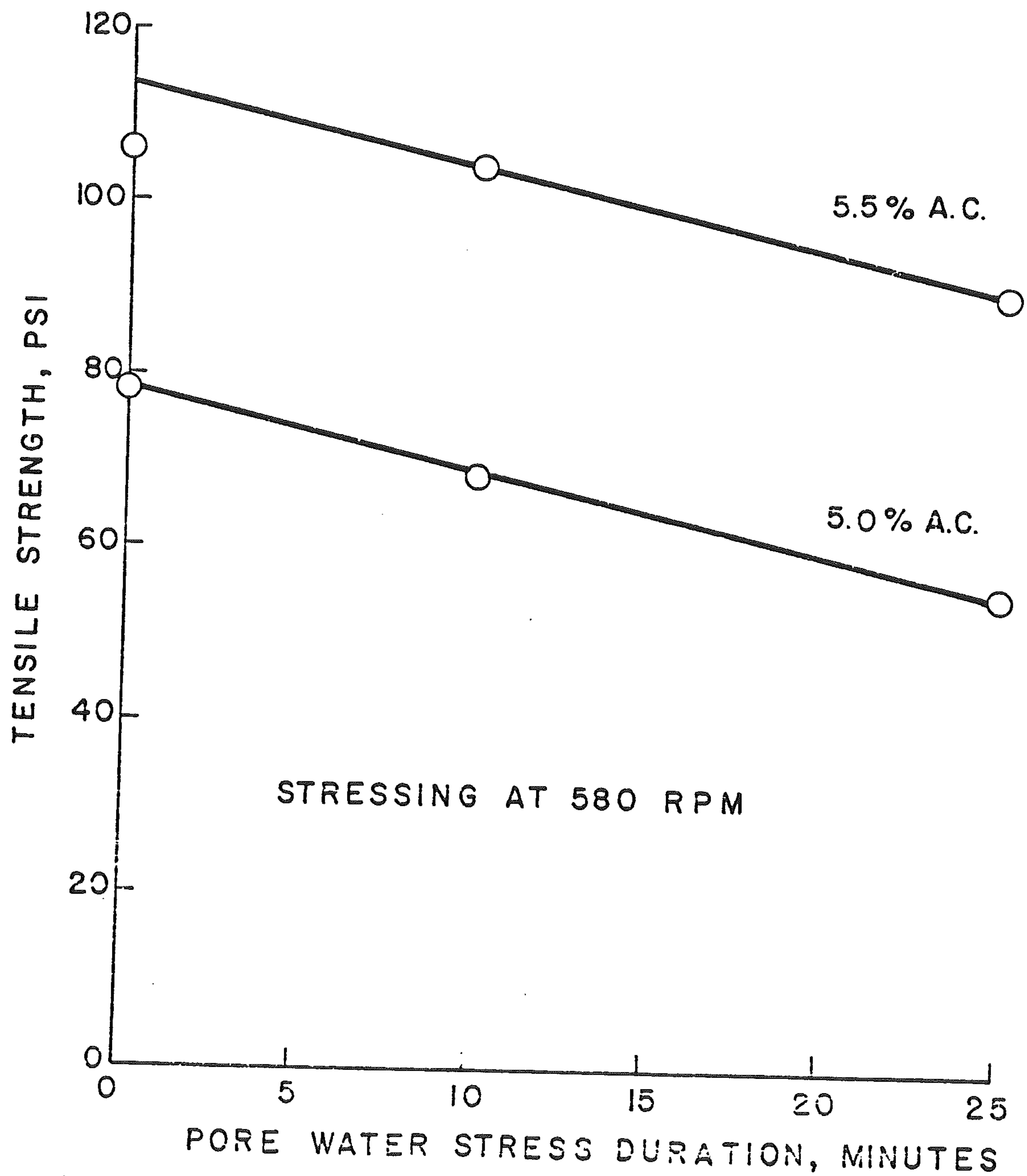

FIGURE 3 Effect of Stress Duration on Tensile Strength for the Tucson Aggregate

Variable Specimen Diameter and Height. It is recognized that the mechanical properties of laboratory prepared specimens are different from those of the same mixture in service. Additionally, 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 from these two sources. The work performed was to establish geometrical effects on the new method of evaluation for stripping susceptibility. The variables involved were asphalt content, specimen height, and specimen diameter; however, due to unforeseen circumstances, the storage time of compacted specimens was an uncontrolled variable. The data for this testing appear in Table 4 of Appendix C. The most acceptable comparison that could be made was that of retained strengths between the 4 and 3.15-inch diameter specimens and it is shown in Figure 4. The indications are that the retained strengths for specimens of these two diameters are essentially the same.

Mixture Variables

The effects of asphalt content and also aggregate cleanliness as described by the sand equivalent value on the retained strength by the new method were determined for the Tucson aggregate. These same variables were used for evaluation with the immersion compression test.

Table C5 in Appendix C shows the results of this testing with asphalt content ranging from 5.0 to 6.0 percent by weight of total mixture and the sand equivalent varying from 31 to 91. The intermediate sand equivalent value of 55 was obtained by blending the "wet" and

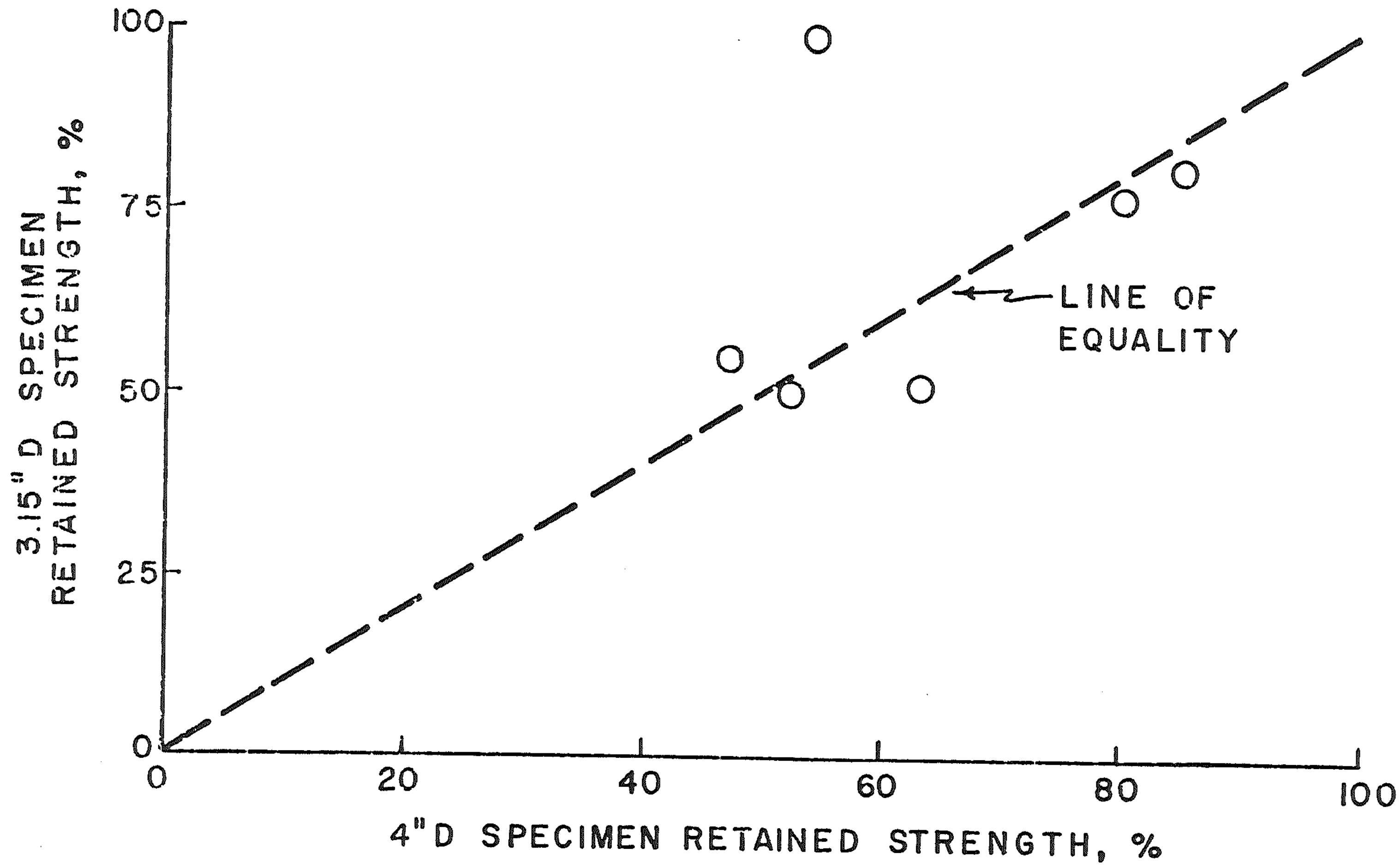

FIGURE 4 Relationship Between the Retained Strength of 4" and 3.15" Diameter Specimens with Variable Asphalt Content

"washed" Tucson aggregates having S.E. values of 31 and 91 respectively. In analysing the data, one must recognize that the gradation of the three blends was not constant.

A review of the data presented in Table C5 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 asphalt content and sand equivalent value increased the retained strength also increased for both methods of testing.

It is noted again from the data of Table C5 and others that the degree of saturation of specimens subjected to the repeated pore pressure stressing can be greater than one hundred percent.

In Figure 5, the curves show the effects of asphalt content and sand equivalent value on the wet and dry strengths obtained by the new method. It is noted that generally the dry strength decreases as the sand equivalent value increases. This behavior could be due to the loss of filler material (-#200) in the blend in order to increase the sand equivalent value. Note also that specimen density also decreases with increases of sand equivalent value. The same figure shows that the wet strength increases with sand equivalent value, which is as one might guess.

The data plotted in Figure 6 show that by the new method the retained tensile strength increases as the sand equivalent value is increased. The point established by sand equivalent value of 91 for the 6.0 percent asphalt mixture appears to be out of line; however, one must recall that for all three values of sand equivalent used there was a slight variation in filler content.

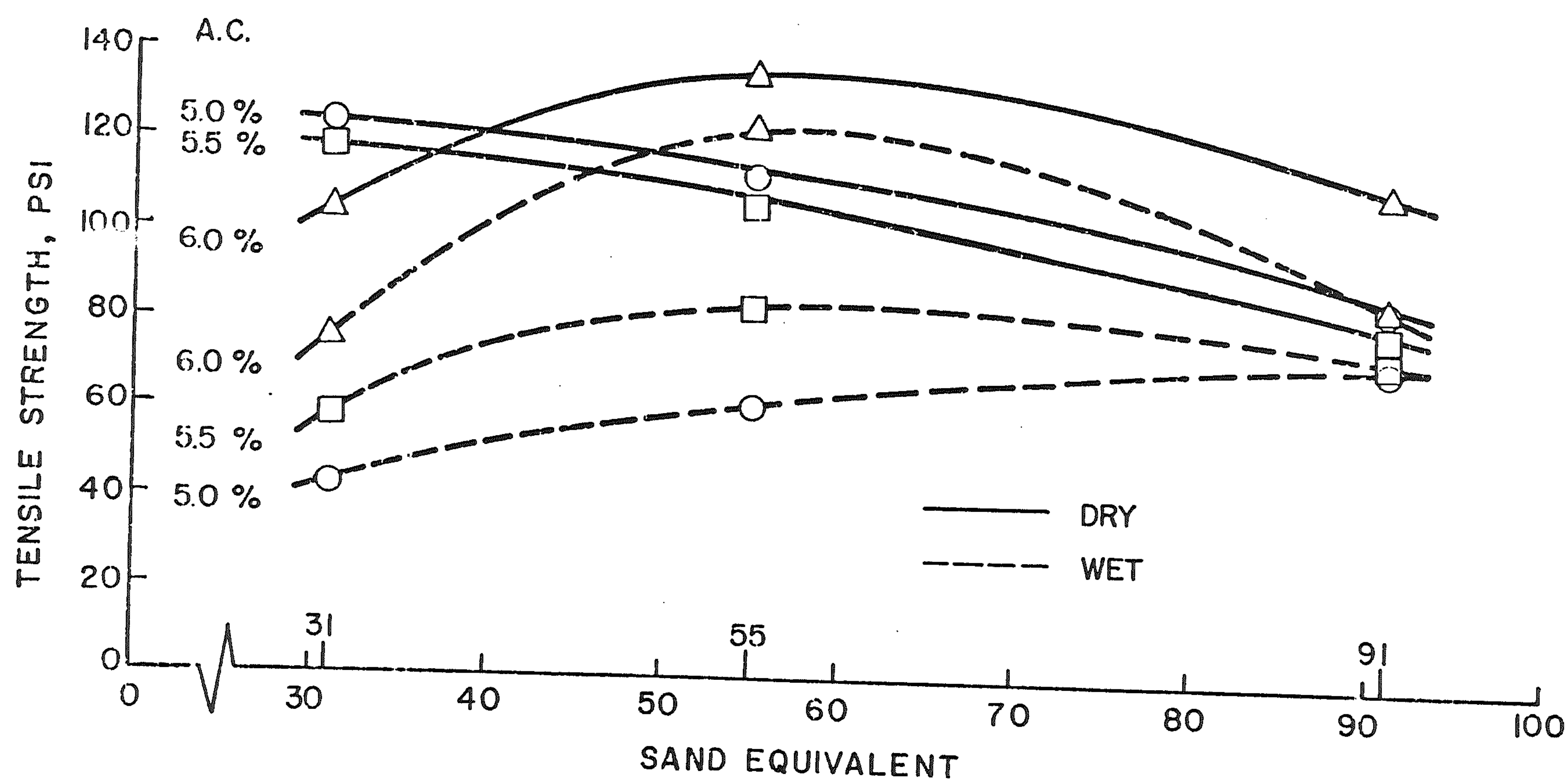

FIGURE 5 Relationship Between Sand Equivalent Value and Tensile Strength as Affected by Asphalt Content for the Tucson Aggregate

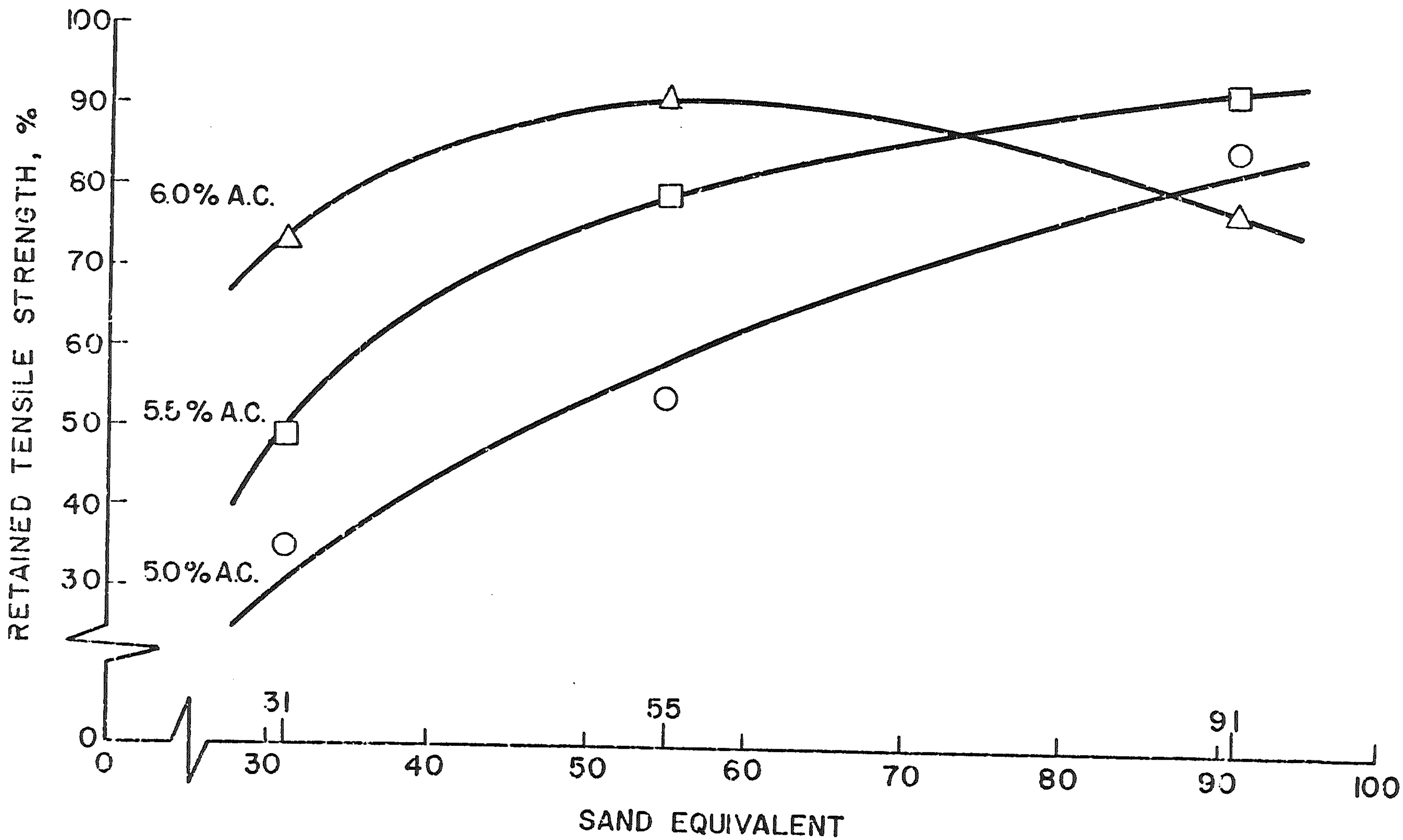

FIGURE 6 Relationship Between Sand Equivalent Value and Retained Tensile Strength for the Tucson Aggregate

Figure 7 presents 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 obtained by the immersion compression procedure. Since the slope of the curve is greater than one, this indicates that the new procedure was more sensitive to the variables than was the immersion compression method. At this time higher value does not mean better.

Limestone Aggregate

As mentioned earlier the Limestone aggregate blend was included in the evaluation program as representing an asphaltic mixture with a good performance record. The evaluation of this blend included variables of sand equivalent value and testing method. Table C6 of Appendix C shows the specimen characteristics for mixtures evaluated by the new and immersion compression methods. The values of particular interest are for the degree of saturation and retained strength. It is apparent that the exposure conditions of both methods result in significant void dilation of the specimens.

The aggregate blend had been presented as one having a good performance record. The data of Table C6 show that the new method yielded retained strengths of 66 and 78 percent for sand equivalent values of 38 and 85 respectively. The immersion compression test yielded retained strengths of 36 and 53 percent for these mixtures.

Holbrook Aggregate (No. 1)

The Holbrook aggregate was included in the work programs to represent an asphaltic concrete with poor resistance to stripping.

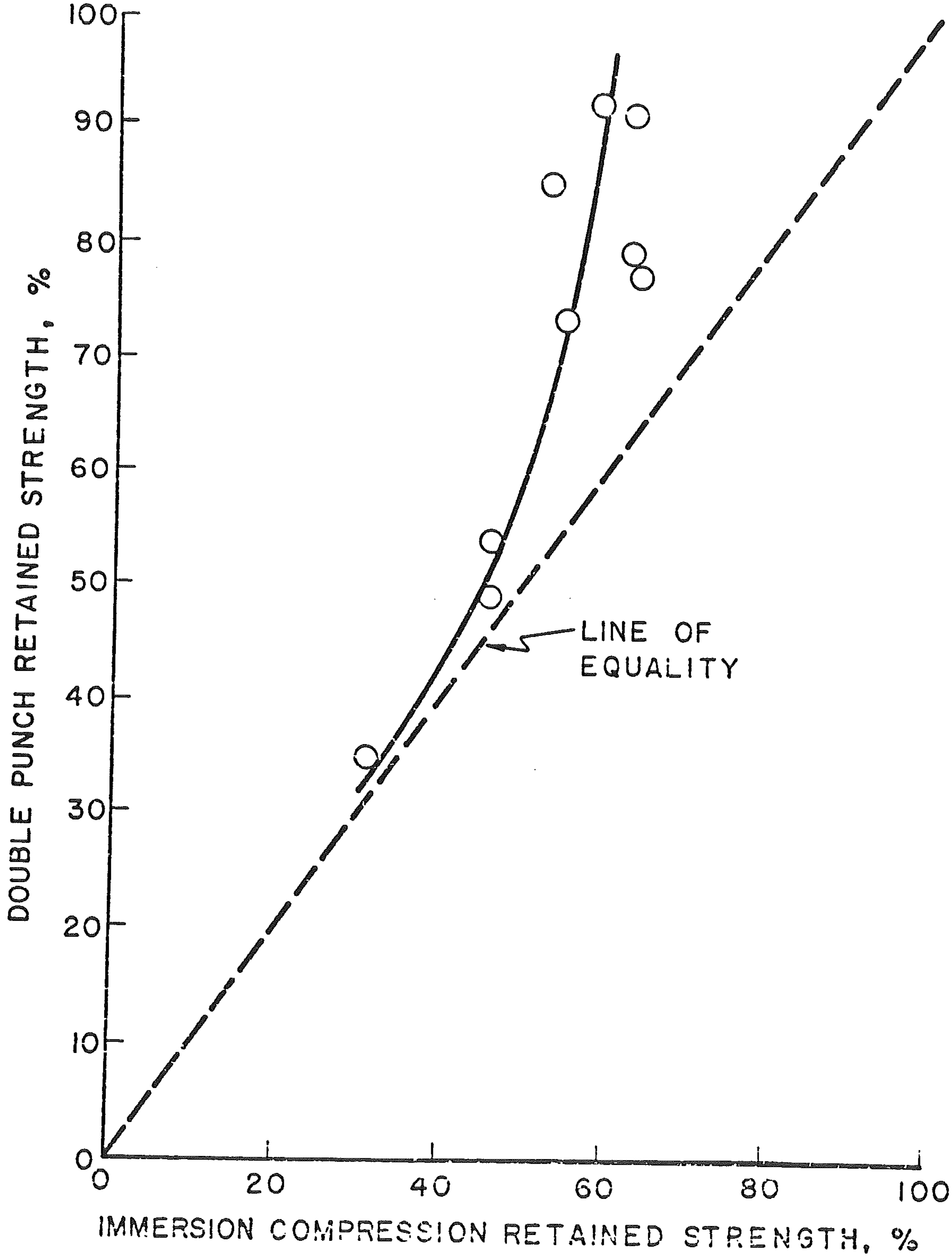

FIGURE 7 Relationship Between Retained Strengths Obtained by the New Method and the Immersion Compression Method for the Tucson Aggregate

Since it was known that the complete aggregate blend as it comes from the pit contained some clay, an elaborate procedure of separating, soaking, elutriating, boiling and final elutriating was utilized to cleanse the aggregate of the deleterious clay. Table C7 in Appendix C lists specimen characteristics for the stripping resistance of asphaltic concrete made with the aggregate. Of particular interest is the relatively high values for degree of saturation for both types of specimen; it appears that aggregate characteristics rather than the exposure conditions are responsible for the values of 150 and about 250 percent for degree of saturation for the immersion compression and double punch specimens respectively.

The difference in sand equivalent value from 59 to 89 did not seem to cause a significant change in retained strength as determined by both methods. Since there was not an appreciable change in gradation caused by the cleansing of the aggregate, the poor resistance to stripping seems to be due to aggregate surface texture, and the composition of the aggregate.

Asphalt Composition

In conversation with personnel of the Arizona Highway Department the thought had been presented that asphalt composition contributed to the stripping susceptibility of an asphaltic mixture. To investigate this thought it was decided to separate the basic asphalt into its Rostler-White components and then recombine 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 and are identified in Appendix B. Since 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 presented in Table C8 of Appendix C. The compositional differences of the asphalts are presented in terms of chemical reactivity ratio, CRR. These data indicate that the CRR value did affect the wet and dry strength separately but there was not a significant difference in retained strength for the various CRR values. A graphical representation of the results for a blend with 5.5 percent asphalt is shown in Figure 8. It is to be noted that generally the wet and dry strengths increased as the CRR value and viscosity at 140°F increased. An analysis of the effects of the various asphalt components on test results is not within the scope of this study.

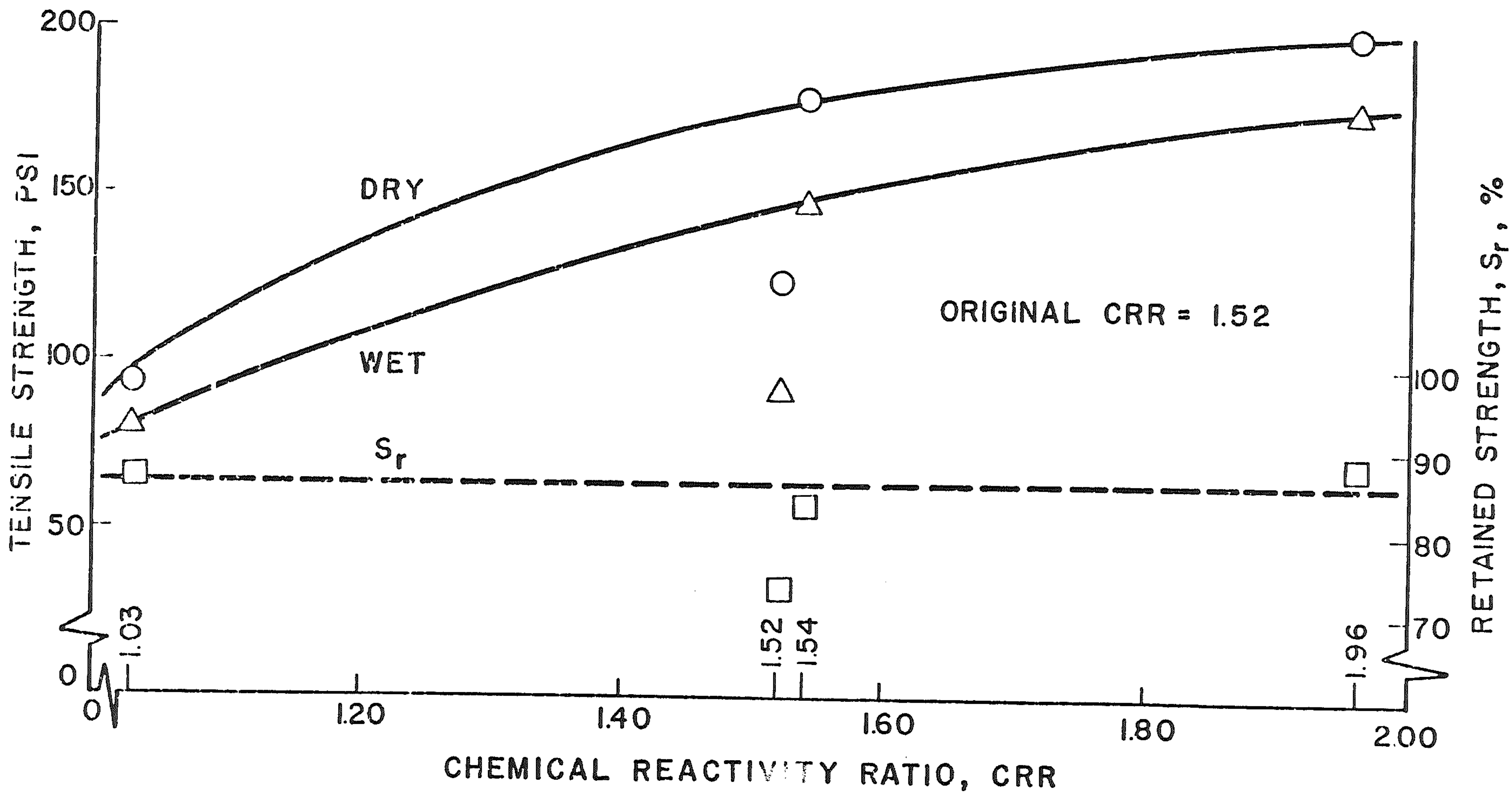

FIGURE 8 Effect of Asphalt Chemical Reactivity Ratio, CRR, on Wet and Dry Tensile Strength for the Holbrook Aggregate No. 2 with 5.5% Asphalt Content

DISCUSSION AND CONCLUSIONS

General

The objectives of this study were to develop an improved method for evaluating the stripping susceptibility of asphaltic concrete. The development has consisted of devising the testing apparatus,procedure, and determining the response of the method to certain factors thought to affect the measurements for resistance to stripping. The work and general results have been described in the preceding sections. The overall evaluation is presented in consideration of the limited amount and exploratory type of work.

Testing Procedure

The general testing procedure appears to be adequate for laboratory evaluation of asphaltic mixtures. The conditions set for saturation, stressing, and strength testing are reasonable with reference to time, temperature, and repeatability. Of particular interest or concern are the measurements for degree of saturation and method for applying the repetitive loads for the exposure of the wet specimens.

There appears to be some relationship between degree of saturation of different aggregate mixtures and their resistance to stripping. It is difficult to accept that in-service paving mixtures dilate under the action of water and/or pore water pressure, nevertheless it is apparent that water-caused 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 be important; however, it is considered that the rate of loading is important in that pore pressure surging and sand particle erosion are affected by cycling rate.

The double punch test was found to be simple and repeatable; however, it suffers as do 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 two 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 indicate that the resistance to stripping was improved with increases in asphalt content and also sand equivalent value. Prior experience indicates that this is an expected result.

Selection of the Limestone and Holbrook aggregates for the study was based on the good performance for the Limestone and poor for the Holbrook. The service record for the Tucson aggregate is considered to have been acceptable for the region. For the natural aggregates 5.5 percent asphalt content seems to be the design amount for the Limestone and Tucson aggregates but it is 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, while the immersion compression test rated the Tucson aggregate as the best and the Limestone as the worst in so far as retained strength is concerned. 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 determined by the new method was surprising. The data show that the higher the CRR value the greater the viscosity and our general experience has been that higher retained strengths are obtained with the immersion compression method for mixtures having asphalt with higher viscosity. In the present case, retained strength 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. It should be noted that 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 obtained coupled with experience in evaluating durability of asphaltic concrete warrant 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 trends that have been established.
3. At the present, specimen and punch sizes must be fixed for comparison of different mixtures to eliminate geometrical effects.
4. As with any new mixture evaluation procedure the new test must be field tested.

Recommendations

The new proposed procedure for evaluating the debonding susceptibility of asphaltic concrete is based on the following precepts:

a. The way to determine the presence of debonding is with a tension test,

b. the action causing debonding in the field should be simulated in the laboratory, and

c. the procedure should be capable of testing a pavement core without the need of remolding.

Since the above considerations are in effect incorporated in the new procedure and since the work presented has indicated acceptable responses with respect to material variables, repeatability, and simplicity, it is recommended that the procedure described in Appendix A be used to evaluate standard Hveems specimens prepared for the routine testing of asphaltic concrete by the Arizona Highway Department. In order to aid in establishing required minimum strength and/or retained strength, in-service mixtures should be sampled on a continuing basis for testing and comparison with initial results obtained from laboratory prepared specimens.

ACKNOWLEDGEMENT

The writer recognizes the contributions of personnel from the Arizona Highway Department and from the Materials and Test Section who obtained the aggregates and also to those who separated and recombined the asphalt for the CRR variation study. In particular, Messrs. Don Stout and Elmer Green were most helpful.

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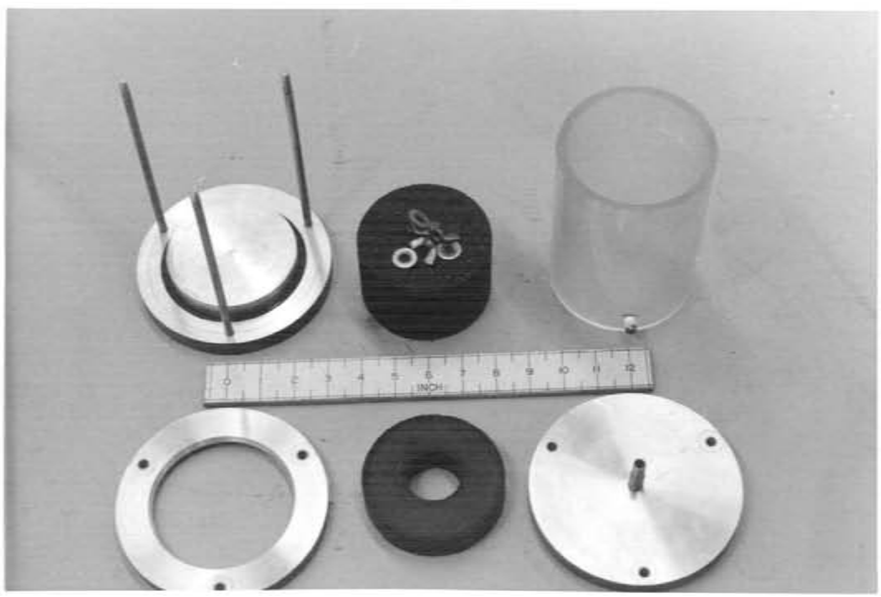
APPENDIX A

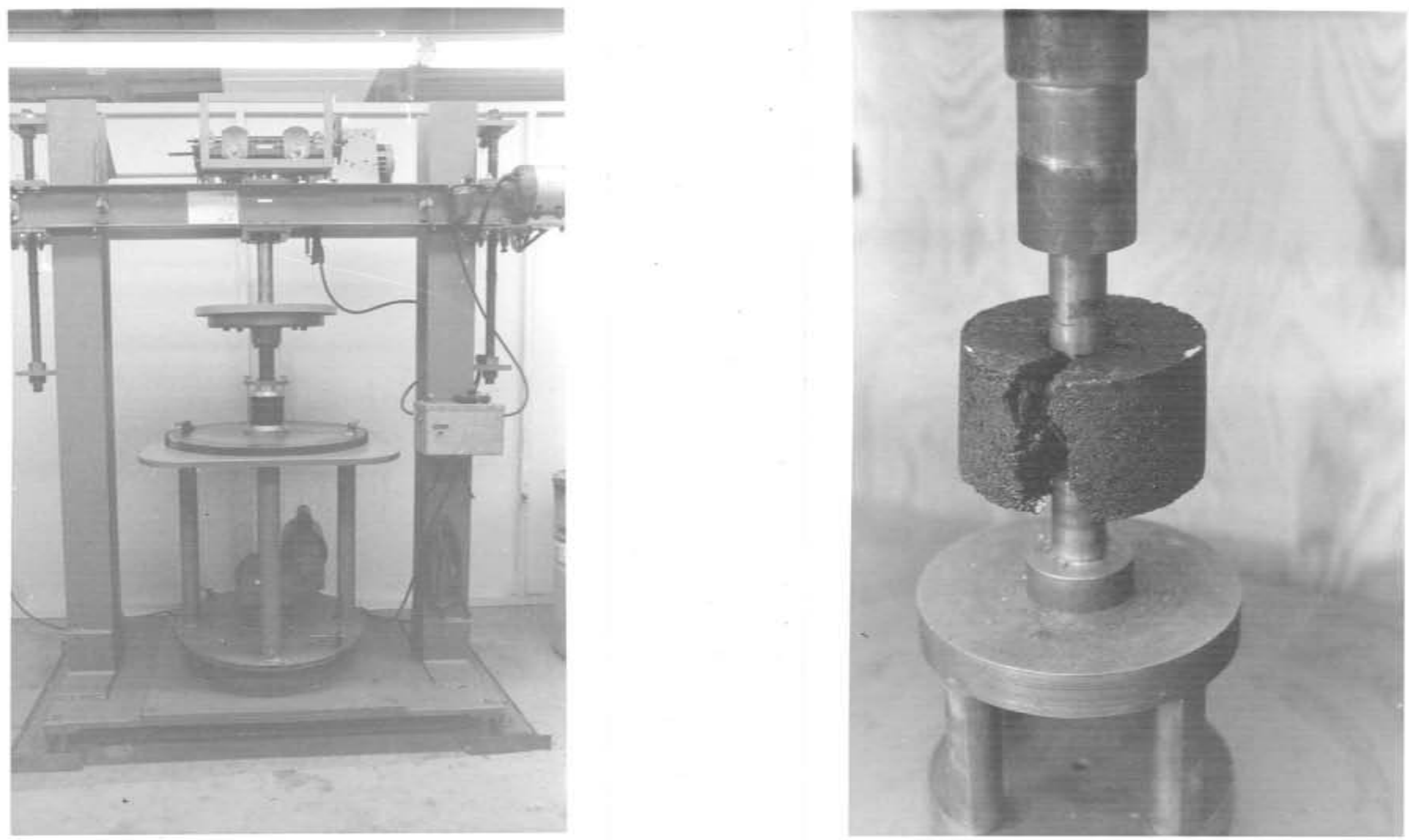
Debonding Test Equipment and Procedure



(a) Disassembled Saturation and Stressing Chamber

(b) Waterbath and Saturation Set-up

Figure A1 Photographs of Vacuum Saturation Equipment



(a) Repeated Pore Water Pressure Stressing

(b) Double Punch Test

Figure A2 Photographs of Stressing and Testing Set-up

DEBONDING TESTS PROCEDURE

Given a 2 1/2" x 4" cylindrical asphaltic concrete specimen, the following basic procedure is followed to determine its resistance to debonding:

a. Saturate Specimen

b. Subject to Cyclic Stressing

c. Test for Strength

Saturate Specimen

Previous testing has shown that the following procedure will yield a high degree of saturation (99% - 100%) in a relatively short time. Water used for saturating specimens is maintained at 122°F (50°C) to a depth of approximately 8" in a water bath; distilled water is preferred. The step-to-step procedure is as follows:

1. Place specimen in the plexiglass saturation/stressing chamber.
2. Place chamber in hot-water bath, cover specimen with about 2" of hot water and secure lid on chamber.
3. Allow to stand in hot water for approximately 15 minutes.
4. Connect vacuum hose to top of chamber and apply vacuum pressure (20" Hg) to chamber for 5 minutes. Spigot at base of chamber should be closed.
5. Release vacuum and let stand an additional 30 minutes in hot-water bath to bring specimen to bath temperature.
6. Specimen is now fully saturated and ready for stressing.

Subject to Cyclic Stressing

The method described below is the result of previous work and is designed so that the specimen is not loaded directly. Loading is accomplished through a layer of water (1/4" to 1/2") between the rubber annulus and the top of the specimen (Figure A3a).

A cyclic load operating at 580 rpm is used to generate pressures within the water saturated specimen ranging from five to thirty pounds per square inch; a range comparable to that expected in saturated pavements subjected to automotiye traffic.

Research has shown that, during the summer months in the desert Southwest, temperatures in saturated asphaltic concrete pavements seldom fall below 122^{o}F. Since it is felt that 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 is attempted in the laboratory by both saturating and stressing the specimen as described above. The following stressing procedure is followed:

1. With the chamber (and specimen) still in the hot-water bath, remove vacuum-tight lid and replace with stressing-ring lid. Secure this lid tightly by hand turning the wing nuts.
2. Place 1" thick Flexane rubber annulus into chamber (beneath 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 1/4" to 1/2" of water remains between its bottom surface and the top surface of the specimen.
4. Remove chamber from hot-water bath and quickly place in proper position on vibratory kneading compactor table.
5. Lower loading apparatus carefully until the 4" diameter foot makes contact with top of annulus. Then continue lowering cross-bar until annulus and water support the weight of the loader.
6. Set timer to the time period required to obtain the desired number of load repetitions and activate the electric motor.
7. After stressing time has elapsed, raise loader, remove chamber from compactor, remove stressing ring lid and annulus. Remove specimen from chamber and place in a 77^{o}F (25^{o}C) water bath for a minimum of 45 minutes.

Test for Strength

After stressing and cooling, the tensile strength of the specimen is determined by the "Double Punch" test method. This test consists of loading with two 1" diameter steel rods (punches) centered on both top and bottom surface of the cylindrical specimen.

If the specimen is to be tested while fully saturated, it is taken directly from the 77^{o}F water bath and tested immediately. If a lesser degree of saturation is required, the specimen is removed from the water and allowed to dry for some period of time before testing. Once the specimen has reached the desired state of saturation, the following test procedure is used:

1. Center specimen on bottom punch using wooden centering blocks.
2. Lower the test machine head until the upper punch just touches the upper surface of the specimen.
3. Set head speed to 1.0" per minute and begin loading. Figure A3b shows a specimen being tested.

The maximum load registered by the testing machine is considered the failure load. The tensile strength is calculated with the following equation:

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

where

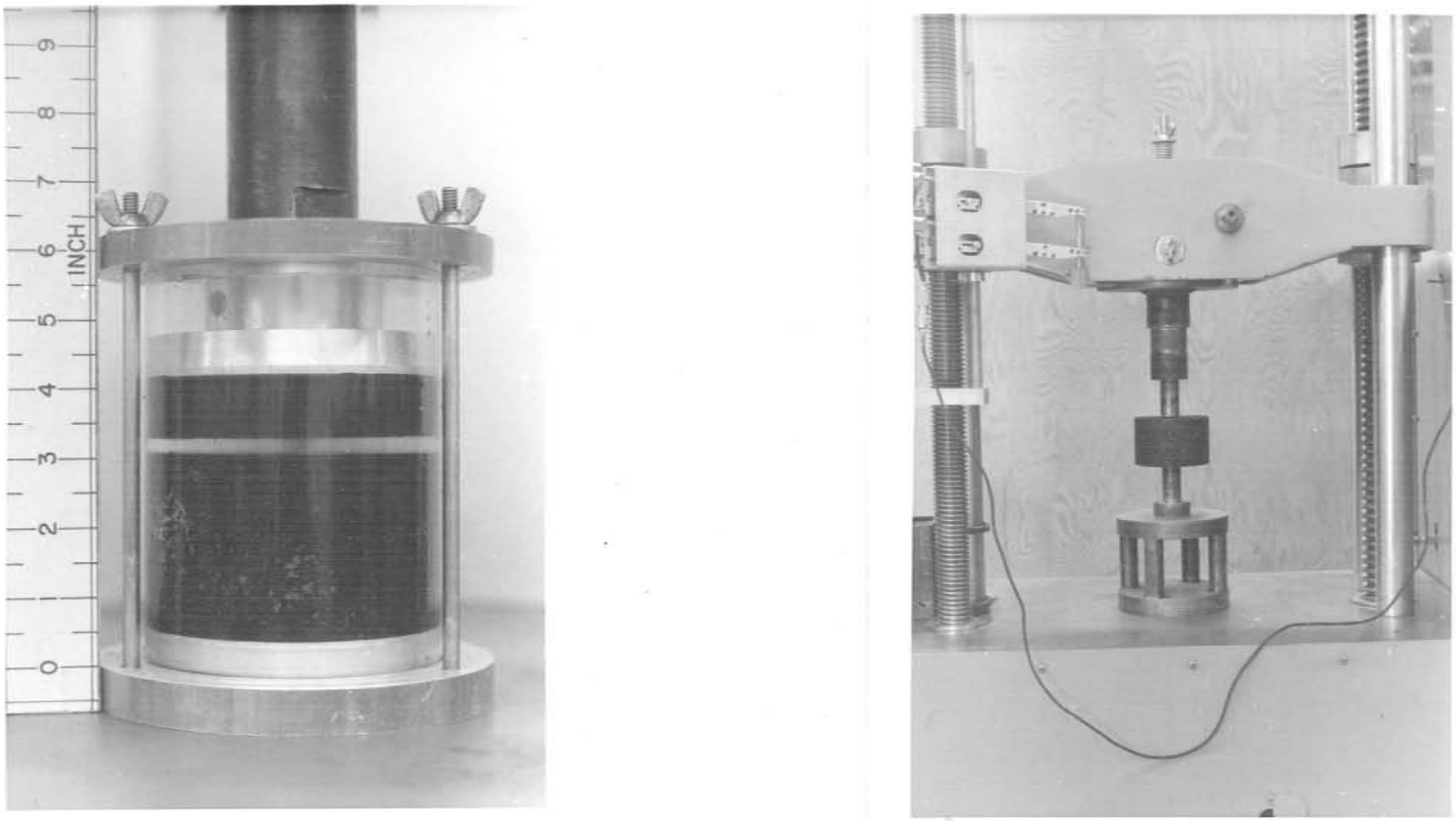
σ_t = tensile stress, psi

P = maximum load, lb.

a = radius of punch, in.

b = radius of specimen, in.

H = height of specimen, in.



(a) Specimen and Rubber Annulus in Stressing Chamber

(b) Double Punch Test

Figure A3 Photographs of Stressing and Testing Set-up

APPENDIX B

Material Characteristics

TABLE B1 Gradation and Sand Equivalent Values of Aggregate Combinations

Gradation

Sieve Size	Percent Passing							
	Tucson		Limestone		Holbrook #1		Holbrook #2	
	Wet	Washed	Wet	Washed	Wet	Washed	Wet	Washed
3/4"	100	100	100	100	100	100	100	100
1/2"	99	99	75	74	87	87	85	85
3/8"	93	93	70	69	73	73	75	74
#4	64	62	59	57	56	56	48	46
#8	44	40	45	43	53	52	43	41
#16	35	31	32	29	45	44	32	30
#30	26	21	22	19	31	30	28	26
#50	17	12	15	11	13	12	14	11
#100	11	5	11	7	3	2	6	3
#200	7	1	5	1	1	0	4	1
Sand Equivalent	31	91	38	85	59	89	--	--

TABLE B2 Component Analysis** and Consistency of Original and Reconstituted Asphalts

Asphalt	Percent of Component					CRR***	Visc. 140° F Poise	Pen. 77° F
	A	N	A_1	A_2	P			
RH	15.81	42.56	13.22	19.81	8.59	1.96	2829	45
RO	18.30	33.20	16.38	21.89	10.23	1.54	1908	67
ORIG.	16.25	29.79	20.79	20.68	12.51	1.52	1895	65
RL	23.75*	26.43	12.33	22.93	14.57	1.03	803	122

* Extra asphaltene added to increase viscosity

** Component analysis by the Rostler-White method (12) and subsequent recombination performed by the Arizona Highway Department.

*** CRR is the chemical reactivity ratio. This expressed as $(N + A_1)/(P + A_2)$.

APPENDIX C

Test Data from Various Testing Programs

TABLE C1 Effects of Variable Storage Time on Tensile Strength of Tucson Aggregate

	Storage Time, Days							
	2		7		42		84	
Asphalt Content, %	5.0	6.0	5.0	6.0	5.0	6.0	5.0	6.0
Density, pcf	141.0	143.0	144.0	145.0	140.0	143.5	139.5	142.5
Voids, %	6.7	3.8	4.9	2.6	7.5	3.7	7.9	4.8
Degree of Saturation, %	128	114	118	121	128	125	119	114
Failure Stress, psi								
Wet*	65	118	125	145	55	98	48	82
Dry	145	149	165	164	143	162	146	141
Retained Strength, %	45	79	76	88	38	61	33	58

* Wet specimens were saturated and stressed 5800 times at 122^{o}F prior to testing at 77^{o}F.

TABLE C2 Effects of Variable Saturation on Tensile Strength After Pore Water Stressing of Tucson Aggregate

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

TABLE C3 Effect of Pore Water Pressure Repetitions on Tensile Strength of Tucson Aggregate*

Number of Stress Applications	Asphalt Content %	Density pcf	Voids %	Degree of Saturation %	Wet Tensile Strength psi
0	5.0	141.0	7.0	95	78
5,800	5.0	141.0	7.0	95	68
13,200	5.0	141.0	7.0	98	55
0	5.5	142.0	5.8	93	106
5,800	5.5	142.0	5.8	93	104
13,200	5.5	142.0	5.8	95	90

* Saturation, stressing and testing at 77°F.

TABLE C4 Effects of Variable Specimen* Diameter and Height on Tensile Strength of Tucson Aggregate

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

* Specimens formed by vibratory kneading compaction.

TABLE C5 Effects of Asphalt Content and Sand Equivalent on Wet and Dry Strengths of Tucson Aggregate Tested by the New and Also Immersion Compression Procedures

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

* In the Double Punch Method, wet specimens are stressed 5800 repetitions at 122^{o}F before testing at 77^{o}F.

TABLE C6 Effects of Sand Equivalent Values on the Wet and Dry Strengths of the Limestone Aggregate Tested by the New and Immersion Compression Procedures

	Asphalt Content, %	
	5.5	
Sand Equivalent	38	85
Double Punch Method		
Density, pcf	141.5	139.0
Void Content, %	4.0	5.8
Saturation, %	158	142
Failure Stress, psi		
Wet*	96	107
Dry	144	137
Retained Strength, %	66	78
Immersion Compression Method		
Density, pcf	139.5	137.0
Void Content, %	6.6	8.1
Saturation, %	135	133
Failure Stress, psi		
Wet	222	199
Dry	620	379
Retained Strength, %	36	53

*In the Double Punch Method, wet specimens are stressed 5800 repetitions at 122°F before testing at 77°F.

TABLE C7 Effects of Sand Equivalent Values on the Wet and Dry Strengths of the Holbrook Aggregate No. 1 Tested by the New and Immersion Compression Procedures

	Asphalt Content, %	
	5.5	
Sand Equivalent	59	89
Double Punch Method		
Density, pcf	144.0	142.0
Void Content, %	2.3	4.0
Saturation, %	281	212
Failure Stress, psi		
Wet*	59	54
Dry	89	95
Retained Strength, %	65	57
Immersion Compression Method		
Density, pcf	138.5	136.5
Void Content, %	6.5	8.5
Saturation, %	154	144
Failure Stress, psi		
Wet	110	94
Dry	274	161
Retained Strength, %	40	58

* In the Double Punch Method, wet specimens are stressed 5800 repetitions at 122°F before testing at 77°F.

TABLE C8 Effects of Chemical Reactivity Ratio, CRR, on the Wet and Dry Strength of the Holbrook Aggregate No. 2

	Original			Reconstituted								
Asphalt CRR		1.52			1.54			1.96			1.03	
Asphalt Content, %	5.0	5.5	6.0	5.0	5.5	6.0	5.0	5.5	6.0	5.0	5.5	6.0
				Double Punch Method								
Density, pcf	148.0	147.0	147.0	147.5	146.5	147.0	146.0	147.0	147.0	146.0	147.5	147.0
Failure Stress, psi												
Wet *	97	92	99	143	148	138	133	176	193	62	80	89
Dry	143	125	109	180	180	173	190	199	199	83	93	85
Retained Strength, %	68	73	91	79	83	80	70	88	97	75	86	107

* In the Double Punch Method, wet specimens are stressed 5800 repetitions at 122^{o}F before testing at 77^{o}F.