Low Temperature Cracking Characteristics of Ground Tire Rubber
And Unmodified Asphalt Concrete Mixture

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ABSTRACT

In recent years, modified asphalt mixtures have become increasingly popular in the construction of flexible pavements. These products have gained popularity because of their ability to increase resistance to rutting at warm temperatures while reducing the occurrence of thermal cracking at cold temperatures. This coupled with the growing problem of waste rubber tires, has led to the reprocessing (grounding) of tire rubber for use in asphalt concrete mixtures.

In order to investigate the low temperature cracking hypothesis, a laboratory research program utilizing, constrained specimen, indirect tension, and direct tension tests, was designed in order to assess the potential benefits of asphalt-rubber concrete mixtures.

Conclusions from this research indicated that the addition of ground tire rubber to asphalt concrete mixtures results in mixtures that exhibit more deformation prior to failure while maintaining similar indirect tensile strength. The research also indicated, through constrained specimen testing, that the addition of ground tire rubber to soft base asphalts (i.e. ACS) resulted in a mixture that exhibited transition and fracture temperatures approximately 10°C (18°F) lower than that of the unmodified mixture.

INTRODUCTION

In recent years, modified asphalt mixtures have become increasingly popular in the construction of flexible pavements. These products have gained popularity because of their ability to increase resistance to rutting at warm temperatures while reducing the occurrence of thermal cracking at cold temperatures. This coupled with the growing problem of waste rubber tires, has led to the reprocessing (grounding) of tire rubber for use in asphalt concrete mixtures.

In order to investigate this hypothesis, a laboratory research program was designed in order to assess the potential benefits of asphalt-rubber concrete mixtures.

BACKGROUND

Low temperature cracking is associated with the volumetric contraction that occurs as a material experiences a temperature drop (1). Materials that are unrestrained will shorten as the temperature drops. However, if a material is restrained, such as the case of asphalt concrete in a pavement structure, the attempt to shorten results in the development of thermal stresses. When these thermal stresses become equal to the tensile strength of the material, a crack is formed.

Asphalt cement, and as a result asphalt concrete, exhibit two coefficients of thermal contraction (1). These are called the glassy and fluid coefficients. The temperature at which the change takes place is called the glass transition temperature. For temperatures warmer than the transition temperature, the asphalt exhibits the fluid coefficient of contraction, while at temperatures colder than the transition temperature the glassy coefficient of contraction is
seen. This indicates that the physical properties of asphalt are significantly different in the fluid or glassy states.

Both asphalt cement and asphalt concrete can be considered to act as viscoelastic materials at warm temperatures (i.e. fluid coefficient) (1). This allows for the dissipation of thermal stresses through stress relaxation. However, at colder temperatures, asphalt concrete behaves as an elastic material and thermal stresses can not be dissipated until a crack initiates (i.e. glassy coefficient). The temperature at which a crack occurs is referred to as the fracture temperature. Once a failure occurs and a crack develops, the stresses are relieved.

In newly constructed asphalt concrete pavements, cracks have been observed to develop 100+ feet of spacing, and as the pavement ages, the crack spacing has been observed to decrease to ten to twenty feet (1).

**Historical Methodology Used in Assessing Thermal Stresses**

Many researchers have attempted to calculate thermal stress of asphalt concrete pavements. Hill and Brien (2) calculated the thermal stresses associated with an infinite, completely restrained strip. The equation used took into account the average coefficient of contraction, initial and final temperatures, and the asphalt mix stiffness (dependent upon time and temperature).

Monismith, et al. (3), used a stress equation developed by Humphreys and Martin (4) to predict thermal stresses in a slab of linear viscoelastic material that was subjected to a time dependent temperature field. The slab was assumed to be of infinite lateral extent and completely restrained. However, in 1969, Haas and Topper indicated that the stresses predicted were unrealistically high (5). They concluded that if Monismith's solution was modified to use a long beam instead of slab, the computed stresses were slightly underestimated (6). This leads back to the approximate solution suggested by Hill and Brien (2), called the pseudo–elastic beam analysis. This solution was supported by Christison and Anderson in 1972 (7), and by two test roads (8,9,10). This methodology was further supported by Finn et al., in 1986 using the cold model to predict low temperature cracking (11).

Thermal stress relationships have also been obtained through indirect estimation. For example, the binder stiffness–temperature relationship at an appropriate (but arbitrary) loading time, may be estimated from the Penetration Index, softening point values and van der Poel's nomogram (1). This binder stiffness–temperature relationship can then be converted to an asphalt mixture stiffness based on the volumetric portions of binder and aggregate present in the mix. Then using an assumed or measured coefficient of thermal contraction, the stress–temperature relationship is obtained using the solution proposed by Hill and Brien (2).

A second form of indirect estimation of thermal stress relationships is based on the load–deformation response of asphalt concrete at cold temperatures. Creep, flexural bending, direct tension, and indirect diametral tension tests have all been used to measure the load–deformation response of asphalt concrete mixtures (3,6,7,12). Previous research has indicated that by multiplying the stress–strain response (load–deformation) of a mixture by the measured or assumed coefficient of thermal contraction, the thermal stress relationship can be estimated.
The development of thermal stresses have also been measured directly in the laboratory (3,13,14,15). This was accomplished by measuring the stress required to maintain a specimen at constant length under a constant rate of cooling. Direct measurement eliminates the need to measure or assume a coefficient of thermal expansion of a mixture.

In 1974, Fabb considered three rates of cooling (5, 10, and 27°C/hr) and concluded that the rate of cooling has little or no effect on the failure temperature (13). However, in 1980, Bloy established that differences in rates of cooling below 5°C/hr did influence the temperature at which cracking occurred in asphalt cements, whereas differences in rates of cooling above 5°C/hr had no influence (16).

State of the Art Methodology Used In Assessing Thermal Stresses

In May of 1990, NCHRP published a procedural manual for design of asphalt concrete mixtures (17). This manual outlines procedures for conducting indirect tensile strength tests and indirect tensile creep tests. The indirect tensile strength test uses a loading rate of 0.05 inches per minute and measures the peak stress obtained. The indirect tensile creep test then conducted on samples using a static load of between five and twenty percent of the indirect tensile strength. The static load is maintained for one hour and then the sample is allowed to rebound for another hour. Vertical and horizontal deformation are monitored throughout the test. The horizontal deformation at the end of the sixty minute load is used to calculate the indirect tensile creep modulus. Both the indirect tensile strength and indirect tensile creep tests are conducted at various temperatures to define the strength-temperature, creep modulus-temperature, and strength-modulus relationships.

The manual then gives an equation which estimates the critical change in temperature at which cracking will occur. This equation is based on the following.

Indirect Tensile Creep Modulus at temperature Ti
Slope and Intercept of Indirect Tensile Creep Curve at temperature Ti
3,600 seconds of relaxation time
Assumed coefficient of thermal contraction between 1.0E-5 and 1.8E-5 in/in/°F

It is also possible to calculate the decrease in thermal stress due to stress relaxation and change in thermal stress due to a drop in temperature using the various combinations of the variables listed above.

Work being completed in the Strategic Highway Research Program (SHRP) contract A00-3A has also addressed the problem of low temperature cracking. Research conducted under this program has addressed the direct measurement of thermally induced stresses on restrained specimens. To date, the results indicate that as the temperature of the specimen is dropped, the asphalt concrete will exhibit stress relaxation down to a certain temperature, a transition temperature, followed by purely elastic behavior. This is shown graphically in Figure 1.

It can be seen from this figure that the slope of the line changes considerably during testing. As the temperature becomes colder, the slope increases until becoming linear. The point at which the slope becomes constant is termed the transition temperature. Above this
temperature the asphalt concrete still possesses viscoelastic characteristics, or in other words, the thermal stresses induced can be relieved through stress relaxation. However, below the transition temperature, the asphalt concrete possesses purely elastic characteristics. The thermally induced stresses are not relaxed until failure of the specimen.

The A003-A researchers have found that the transition temperature is dependent upon mixture properties such as air void content. As the air void content increases the transition temperature decreases. The transition temperature has also been found to be related to the fracture temperature of the mixture. As the transition temperature decreases, so does the fracture temperature. This is shown graphically in Figure 2.

All of the research to date is indicating the lower the transition temperature of the mixture, the better the mixture will perform when considering low temperature cracking. This idea is supported by the stress relaxation (viscoelastic behavior) that is seen when above the transition temperature. Based on this hypothesis, measurement of both the fracture strength and the transition temperature is necessary for proper characterization of low temperature properties.

The scope of this research program included one aggregate source, one gradation and five binders. The test matrix is shown in Table 1.
MATERIALS

Aggregates

The aggregates used in this research program were obtained from Granite Rock Company, located in Watsonville, California. This material is a 100 percent crushed granite that has no history of stripping problems with in-service pavements. The physical properties of the aggregate are shown in Table 2.

The gradation used to prepare the mixture samples is shown in Table 3. This gradation was chosen to meet ASTM D3315 1/2" dense mixtures, Nevada Type II and California 1/2" medium specification (Table 3). This gradation was opened up slightly on the #30 and #50 to accommodate the presence of rubber.

Binders

The two grades of neat asphalt cement used in this phase of the research program were obtained from a single California Valley crude source. The binders used were:

Unmodified: AC5
- AC20

Both the AC5 and AC20 were then modified with crumb rubber. The AC5 was also modified with rubber and extender oil, yielding a very soft third modified binder. The source of crumb rubber was selected by the sponsor with the rubber being blended with the asphalt cement by Crafco Inc., located in Chandler, Arizona. The rubber used in this research program was ambient ground rubber having a hydrocarbon content of approximately forty-five percent and a specific gravity between 1.100 and 1.200. The particle size, along with the gradation specification suggested by Crafco are shown in Table 4. The resulting modified binders were:

Modified: AC5 + 17% Rubber (AC5R)
- AC5 + 16% Rubber + 5% Extender Oil (AC5RE)
- AC20 + 16% Rubber (AC20R)

OPTIMUM BINDER CONTENTS

In phase 1 of this research, binder contents to be used in phases 2, 3, and 4 were selected by a committee that included the sponsor and all of the researchers involved. These selections were based on mix designs conducted at both the University of Nevada, Reno and the U.S. Army Corps of Engineers, Waterways Experiment Station (WES). Optimum binder contents for both unmodified mixtures, AC5 and AC20, were agreed upon at 5.3 and 5.7 percent by total weight of mix, respectively. However, there was disagreement as to the binder content to use for each of the modified mixtures. As a result, a compromise was made that was agreeable to all parties involved in the extended program. The compromise yielded binder contents that were higher than the UNR recommended optimums. The following table shows the binder contents used and the UNR recommended binder content for all modified mixtures.
<table>
<thead>
<tr>
<th>Type of Binder</th>
<th>Binder Content Used In Preparing Samples (% Total Weight of Mix)</th>
<th>UNR Recommended Binder Content (% Total Weight of Mix)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC5R</td>
<td>8.5</td>
<td>7.7</td>
</tr>
<tr>
<td>AC5RE</td>
<td>8.3</td>
<td>7.7</td>
</tr>
<tr>
<td>AC20R</td>
<td>7.9</td>
<td>7.4</td>
</tr>
</tbody>
</table>

**SAMPLE PREPARATION**

Samples were batched by first separating the aggregates into the eleven individual sizes (1/2", 3/8", 1/4", #4, #8, #16, #30, #50, #100, #200, fines) needed to prepare samples, and then recombined to meet the desired gradation. Washed sieve analysis were performed on complete batches to ensure the gradation had been met.

After all aggregate preparation was completed, batches were selected at random and mixed with the selected binder. Different methods of mixing were used for the asphalt-rubber and unmodified mixtures. Samples were also compacted to achieve approximately six to eight percent air voids. This resulted in using different levels of compaction for the various mixtures used in this research program. The procedures for mixing and compaction are described below.

Unmodified mixtures were mixed in accordance with ASTM D 1561 (20). After mixing, samples were placed in a 140°F forced draft oven for fifteen hours prior to being reheated to 230°F for compaction. Rubber mixtures were mixed using the recommendation of Chehovits (21). This involves heating the aggregate to 300°F and the asphalt-rubber binder, regardless of base asphalt viscosity, to 350°F prior to mixing. Once again, after mixing, samples are placed in a 140°F forced draft oven for fifteen hours prior to reheating the samples for compaction.

All samples were compacted with a kneading compactor. Two types of samples were prepared for testing under this phase of the research, normal 2" in height in 4" in diameter briquets and 3" deep by 3" wide by 16" long beams.

Unmodified briquets were compacted using 30 blows at 250 psi. This was followed by curing for 1-1/2 hours at 140°F prior to the application of an 11,000 lb leveling load. Samples were then allowed to cool before being extruded.
Asphalt-rubber briquets using AC20R were reheated to 300°F for compaction, while the other two rubber mixtures (AC5R and AC5RE) were reheated to 230°F for compaction. Compaction of all asphalt-rubber briquets consisted of 30 blows at 250 psi. Briquettes of AC20R were cured in a 140°F oven for 2 hours prior to the application of an 11,000 lb leveling load. Samples of AC5R and AC5RE were cured in a 140°F for 1-1/2 hours prior to the application of an 11,000 lb leveling load. All samples were allowed to cool and were then extruded.

Unmodified beam specimens were compacted in two lifts. The first lift consisted of two thirds of the material needed for the beam and received twenty blows at 75 psi. The second lift consisted of the other third of the material. The specimen then received forty blows at 75 psi, forty blows at 100 psi, and forty blows at 200 psi. This was immediately followed by level loading to 10,000 lbs. Beam specimens were then allowed to cool prior to extruding.

Asphalt-rubber beam specimens, regardless of binder type, were reheated to 230°F prior to compaction. These beam specimens were also compacted in two lifts, with the first lift consisting of two thirds of the mixture needed. This lift received twenty blows at 75 psi. The second lift, consisting of the other third of the material needed, was then placed in the mold. The specimen then received forty blows at 75 psi. The completed specimen then was level loaded immediately to 10,000 lbs. Specimens were then allowed to cool before being extruded. The low amounts of compactive effort needed to fabricate asphalt-rubber beams is due to the dilatent (shear thinning) characteristics of the asphalt-rubber. The kneading action used in producing beams imparts a large amount of shear to the mixture. This resulted in relatively low amounts of needed energy to compact asphalt-rubber beams to the appropriate air void content.

After all beam specimens, asphalt-rubber and unmodified, were allowed to cool overnight in a 77°F room, they were sawed to 2 inches in depth by 2 inches in width by 10 inches in length for testing. This was done to provide cut surfaces on each face of the sample, in order to remove the irregularities that are associated with laboratory compacted samples.

Selected samples from both briquettes and beams were subjected to accelerated laboratory aging in order to assess the effects of aging on both the unmodified and asphalt-rubber mixtures. The aging method used is consistent with NCHRP 9-6(1) AAMAS. This method consists of subjecting compacted briquette and beam, in this case sawed specimens, to forty eight hours of forced draft oven heating at 140°F followed by 120 hours of forced draft oven heating at 225°F. Samples were then cooled to testing temperature and tested according to the appropriate testing sequence.

TESTING METHODS

Following compaction, saw cutting, and/or aging, samples were allowed to cool in a 77°F room prior to being tested for bulk specific gravity and height. All heights were determined in accordance with ASTM D3515 (20). Bulk specific gravity of compacted briquets was conducted in accordance with ASTM D2726. The bulk specific gravity of sawed beams was determined using a modified version of the paraffin coated procedure. This procedure uses a removable film called “Parafilm”. This paraffin based film is commercially available and was used to
ensure that no water was being absorbed into the cut surfaces of the aggregate. This test method is currently being addressed by ASTM for an alternate to D1188 (paraffin). Samples were then placed under a fan, again overnight, to remove any moisture that may have penetrated the sample during bulk specific gravity testing. Samples were then placed in an appropriate temperature control chamber to condition them to the testing temperature. After a minimum of twenty four hours in that particular temperature control chamber, samples were tested for low temperature cracking characteristics using one of three methods. The procedures used for these testing methods are described below.

The first of three tests used to assess low temperature cracking was a constrained specimen test. Sawed beams were glued to platens that connected to the loading frame with universal joints. Universal joints were used in order to remove any eccentricity that may result from the gluing process (Figure 3). Testing started at \( T^\circ C \) with the sample held at a constant length through the use of a closed loop testing system. The temperature in the chamber was dropped at a rate of 10°C per hour. The resulting load, induced by the sample trying to shrink, was measured constantly and recorded every minute. The temperature on the surface of the specimen was also monitored throughout the test, allowing a temperature versus load relationship to be obtained for each specimen tested. The test was considered to have ended when a sample fails in a brittle manner or thirty minutes after peak load in the case of ductile failure.

The second test method used to assess low temperature cracking was the indirect tension test. This test used the conventional size briquets (i.e. 2" by 4") and tested them in the diametral position. A tensile load was achieved by applying a compressive load across the diameter of the specimen, parallel to the height Figure 4. Samples of each mixture were tested at 34°F, 0°F, and -20°F. A constant loading rate of 0.01 inches per minute was used for all tests. The peak load for each specimen was recorded for analysis.

The third and final test method used to assess low temperature cracking was a direct tension test. This test also made use of the sawed beams. Specimens were glued to platens and mounted to the loading frame in a manner consistent with that used to test constrained specimens. Samples were tested at -20°F using a constant loading rate of 0.01 inches per minute. Once again, the peak load achieved during testing was recorded.

**TESTING PROGRAM**

A total of 99 samples were prepared for testing. This allowed for three replicates to be tested under each testing condition. The testing matrix is shown in Table 1. The replicate samples tested provided sufficient data to estimate the mean, standard deviation, and coefficient of variation for each type of mixture tested for each condition.
ANALYSIS OF TEST RESULTS

As stated previously, there were three types of low temperature cracking tests used in this research program. For ease of discussion, the analysis will be presented in the same fashion, first the constrained specimen, followed by the indirect tension, and finally the direct tension test results.

Constrained Specimen Test Results

As stated previously, the data derived from constrained specimen testing consists of the induced tensile load versus temperature relationship. From this relationship it is possible to retrieve the transition temperature (Tt), the fracture temperature (Tf), and the peak load (i.e. load just before fracture).

Figure 5 shows the average induced tensile load versus temperature relationship for mixtures using the unmodified ACS (individual test results are shown in appendix A). It can be seen from this figure that average transition temperature is approximately \( -12^\circ C (10^\circ F) \). Table 5 shows the individual data for peak load and fracture temperature as well as the average, standard deviation, and coefficient of variation (CV) for each, for samples using ACS. This table indicates that an average peak load of 287 psi was achieved and that the average fracture temperature was \(-26^\circ C (-15^\circ F)\). CV's for the peak load and fracture temperature are 7.3% and 9.7%, respectively. This data indicates that the test results show very good repeatability.

Figure 6 show, the average induced tensile load versus temperature relationship for mixtures using the unmodified AC20 (again, the individual test results are shown in appendix A). This figure shows an average transition temperature of approximately \(-11^\circ C (12^\circ F)\). Table 6 shows the individual data for peak load and fracture temperature as well as the simple statistics that are shown in Table 5. This table shows an average peak load of 278 psi and an average fracture temperature of \(-25^\circ C (-13^\circ F)\). Once again, the test repeatability is very good (CV's of 11.2% for the peak load and 4.3% for the fracture temperature). It is interesting to note the all three types of data, Tt, Tf, and peak load, are approximately the same for both grades of unmodified asphalt. This leads to the hypothesis that the controlling factor might be the source of the crude petroleum used in refining the asphalt cement.

Figure 7 shows the average induced tensile load versus temperature relationship for the asphalt-rubber ACS (ACSR). The figure indicates a transition temperature for the ACSR of approximately \(-22^\circ C (-8^\circ F)\), considerably lower than the unmodified ACS. Table 7 shows the individual test results and simple statistics for the ACSR. This table shows an average peak load of 237 psi and an average fracture temperature of \(-34^\circ C (-29^\circ F)\). It can be seen that the CV for the peak load remains very low, at 8.9%, while the CV for the fracture temperature jumps to 20.8%. This is somewhat higher than would be hoped for, but is still in the range of acceptable data.

The average induced tensile load versus temperature relationship for the asphalt-rubber AC20 (AC20R) is shown in Figure 8. This figure signifies a transition temperature of \(-14^\circ C (7^\circ F)\). After seeing the reduction in transition temperature for the ACSR, the AC20R data is somewhat disappointing. Table 8 shows individual data for peak load as well as the simple statistics used in earlier tables. It can be seen that the average peak load is 157 psi and the average fracture temperature is \(-25^\circ C (-13^\circ F)\). Inspection of the CV for each type of data
indicates that peak load is consistent, \( CV = 12.7\% \), but that the fracture temperatures obtained from the test are extremely undesirable, \( CV = 45.8\% \). It is hypothesized that the rubber swells during mixing due to the absorption of the lighter weight molecular particles of the asphalt cement. This tends to leave the asphalt phase of the binder system with the heavier oil and resins. This could result in an increase in binder viscosity and hence mixture stiffness. Also, the heterogeneity of the composition of the failure some cross section could be contributing to the increased testing variability (i.e. increases in CV). This is due to the random and not always uniform distribution of the rubber phase of the binder system.

This could explain to some extent the increase in the fracture temperature CV for the AC5R. It would also explain the conclusion that the addition of rubber to an AC20 to improve its low temperature cracking properties is not a good approach. A softer binder should be used, preferably one that will provide enough light ends to have a percentage of them remaining in the asphalt cement phase of the binder system.

According to Table 1, three samples of ACSRE were to be tested using the constrained specimen test. This testing was unable to be completed because the beam specimens using this extremely soft binder literally fell apart with any sort of handling.

**Indirect Tension Test Results**

Data obtained from the indirect tension testing consisted of the peak load achieved during testing. Understanding that the peak load alone does not give a complete picture of material stiffness, the approximate testing time was also recorded. This data will help to give a more complete picture of how the various mixtures behave under diametral loading.

Figure 9 shows the average indirect tensile strength for all five mixtures tested at 34°F (1°C). Visual inspection shows both of the unmodified mixtures are considerably stronger than any of the asphalt-rubber mixtures. Inspection of Table 9, which shows individual test results as well as the average, standard deviation, and coefficient of variation, indicates that there is overall good repeatability.

Figure 10 shows the average indirect tensile strength for all five mixtures tested at 0°F (-18°C). This figure shows that all three mixtures using the ACS (ACS, AC5R, and ACSRE) are approaching equal tensile strength. This is regardless of the presence of rubber or extender. The figure also indicates that at this temperature the AC20R mixture is stiffer than the AC20 mixture. Table 10 shows that the repeatability (CV) has dropped considerably from the 34°F test results. This loss of repeatability hampers the ability to make firm conclusions, however, general trends can still be identified. In general, a trend of the asphalt-rubber mixtures nearing or exceeding the tensile strength of the unmodified mixtures.

It is interesting to note that unmodified mixtures took approximately ten to twelve minutes of loading to achieve failure while the asphalt-rubber modified mixtures needed approximately twenty minutes of loading to achieve failure. These times to failure are only approximate, however, they do indicate that the asphalt-rubber mixtures exhibit more deformation prior to failure.
Figure 11 shows the average indirect tensile strength for all five mixtures tested at -20°F (-29°C). Visual inspection of this figure shows very little difference between any of the mixtures. This indicates that all five mixtures are exhibiting “glass” characteristics. Once again, however, the asphalt-rubber mixtures required more loading time to achieve failure than the unmodified mixtures. The asphalt-rubber mixtures required approximately ten to twelve minutes to fail while the unmodified mixtures required approximately five to seven minutes to fail. This again, signifies that the asphalt-rubber mixtures failed in a more ductile manner than the unmodified mixtures. Table 11 shows the individual data, average, standard deviation, and coefficient of variation for all five mixtures tested at -20°F (-29°C). This table indicates that there is very good repeatability within each mixture. This leads to the conclusion for this test method, conducted at -20°F (-29°F), the mixtures exhibit similar tensile strength but the rubberized mixtures are the more ductile material.

Figure 12 compares the indirect tensile strength test results at all three temperatures. This figure shows the average indirect tensile strengths for each of the five mixtures at all three testing temperatures. It can be seen that at 34°F (1°C) the asphalt-rubber mixtures are softer than the unmodified mixtures, but that at 0°F (-18°C) the mixtures approach equal strength. Test results at -20°F (-29°C) indicate the two mixtures incorporating AC20 peaked in strength at 0°F (-18°C). The other three mixtures are still exhibiting increasing strength. This suggests that the AC20 and modified AC20 (AC20R) will be the most brittle at cold temperatures.

Direct Tension Test Results

Data obtained from the direct tension testing also consisted of the peak load achieved during testing. Figure 13 shows the average test results from all five mixtures tested at -20°F (-29°C). Visual inspection shows very little difference between any of the mixtures. Inspection of Table 12 shows the test repeatability is acceptable. Ideally, the CV for each mixture would be below 15%, however, the range obtained from this testing can still be used to make conclusions about the data. The conclusion here is that the extremely low temperature properties of each mix will be similar in as much as they all contain the same supplier of asphalt cement.

Efforts were made to see if there was a correlation between tensile strength determined from indirect method and from the direct method. Figure 14 shows the average tensile strength values from all five mixtures for both the indirect and direct testing methods. The data indicates that there is very little difference between the data derived from the tests. This is most likely due to the fact that all mixtures are behaving in a similar manner at this temperature (-20°F or -29°C). This leaves no real basis to compare the tests themselves. In order to properly assess the differences between the tests, there would need to be differences from mixture to mixture.

Analysis of Aging Sample Test Results

Table 1 indicates that there were samples prepared for constrained specimen, indirect tension, and direct tension testing that were to be subjected to accelerated oven aging prior to testing.

This figure indicated that three samples each of AC5, ACSR, and AC5RE were to be tested using the constrained specimen method. Unfortunately, all beam samples suffered some sort of
crumbling of the cut surface during the aging process. Attempts were made to cut the specimens to lengths that contained no damage, however, the data obtained from these efforts was unusable. This leads to the conclusion that cut beams, in particular those containing a relatively soft base asphalt, will tend to crumble at the elevated temperatures used in the NCHRP AAMAS accelerated aging process. It should be noted that these problems were not encountered with the briquettes. Therefore, the accelerated aging testing was completed on the aged briquettes with no problems encountered.

Table 1 indicates that three samples of the mixture using ACS were to be tested at 0°F (-18°C) utilizing the indirect tension test method. The average indirect tensile strength from this testing is shown along side the unaged ACS test results in Figure 15. It can be seen from this graph that there is approximately a twenty five percent increase in indirect tensile strength after the oven aging. This is further supported by the low coefficient of variation for this testing (Table 13).

Three samples of mixture using the ACSR binder were also tested for indirect tensile strength at 0°F (-18°C). The average indirect tensile strength for this testing is shown along side the unaged test results in Figure 16. This data also shows an increase in indirect tensile strength of approximately twenty five percent. However, this data showed very poor repeatability (CV of 40.4%). Therefore, it is hard to draw any tangible conclusions.

Figure 17 shows the average indirect tensile strength for both aged and unaged samples of mixture using ACSRE for all three temperatures used for the indirect tension testing. Inspection of this figure shows that for both the 34°F (1°C) and 0°F (-18°C) testing temperatures, there is approximately a seventy percent increase in tensile strength from unaged to aged test results. Data obtained at -20°F (-29°C) indicates a tensile strength increase of approximately thirty five percent. It is hypothesized that at the two warmer testing temperatures, the asphalt-extender phase of the binder system is absorbing the load, and not the rubber. Therefore the largest increases in tensile strength are noticed at these temperatures because the extender oil and the light ends of the asphalt cement are the first to be cooked off during the aging process. At -20°F (-29°C) the rubber phase of the binder system is absorbing the load, and thereby reducing the effects of the asphalt-extender phase of the system. This reduction in effect leads to lower increases in strength due to aging.
CONCLUSIONS

Based on the analysis presented in this paper, the following conclusions are offered.

1. Constrained specimen testing yielded approximately equal values of transition temperature, fracture temperature and peak load for both unmodified mixtures (ACS and AC20). This could be due to the inability of the test to distinguish these properties from asphalts of the same source.

2. Constrained specimen testing indicated that mixtures using the asphalt-rubber AC5 binder produced transition and fracture temperatures approximately 10°C (18°F) lower than mixtures using the unmodified ACS.

3. Rubber particles swell during mixing due to the absorption of the lighter weight particles of the asphalt cement. This tends to leave stiffer base asphalt cements with only the heavier oils and resins. This could result in an increased binder viscosity and hence mixture stiffness. Mixtures exhibiting this increase in stiffness become very sensitive to any non-homogeneities in the mixture. This sensitivity results in increased testing variability due to the random and not always uniform distribution of the rubber particles in the binder system.

4. Softer base asphalts should be used in rubber modified systems for low temperature thermal cracking applications. Preferably one that will provide enough light ends to leave a percentage of them in the asphalt cement phase of the binder system.

5. Indirect tension testing indicated that rubber modified mixtures will exhibit more deformation at colder temperatures (i.e. 0°F and -20°F) while maintaining strengths similar to unmodified mixtures.

6. Cut beam specimens, in particular those containing a relatively soft base asphalt, will tend to crumble at the elevated temperatures used in the NCHRP AAMAS accelerated aging process. It should be noted that these problems were not encountered with the briquette specimens.

7. The addition of rubber to mixtures using AC5 did not change the tensile strength characteristics of the mixture with aging. Both mixtures exhibited approximately a twenty five percent increase in indirect tensile strength after aging. However, it should be noted that the aged ACSR mixture tensile strength was slightly less than the unaged AC5 mixture.

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BIBLIOGRAPHY


