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# Investigation of Low Temperature Thermal Cracking in Hot Mix Asphalt

Tim Aschenbrener
Colorado Department of Transportation
4340 East Arkansas Avenue
Denver, Colorado 80222

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U.S. Department of Transportation
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16. Abstract

A study was performed to determine the influence of material properties on the thermal cracking performance of hot mix asphalt (HMA), and to determine the ability to predict thermal cracking from pavements of known field performance. The testing device used to measure the HMA properties was the thermal-stress, restrained-specimen test (TSRST), and the device used to measure the binder properties was the bending beam rheometer (BBR).

The laboratory study was conducted to determine the variability of test results as an influence of 1) asphalt cement stiffness, 2)asphalt cement quantity, 3)mixes with various aggregate qualities, 4)aging, and 5)the presence of hydrated lime. The influence of the asphalt cement stiffness was the single largest factor that controlled the test results.

The field study was performed with 9 sites of known thermal cracking performance. Correlation of the test results to the known field performance was not very good. The best correlation was with the fracture strength measured by the TSRST and the m-value from the BBR.

For samples prepared and aged in the laboratory, the BBR results on the binder gave approximately 2°C warmer temperatures than the TSRST results on the mix. For field samples, the BBR results on the binder gave approximately 2°C cooler temperatures than the TSRST results on the mix.

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**Investigation of the Low Temperature Thermal Cracking in Hot Mix Asphalt**

Tim Aschenbrener

# 1.0 Introduction

In September 1990, a group of individuals representing AASHTO, FHWA, NAPA, SHRP, AI, and TRB participated in a 2-week tour of six European countries. Information on this tour has been published in a "Report on the 1990 European Asphalt Study Tour" (*1*). Several areas for potential improvement of hot mix asphalt (HMA) pavements were identified, including the use of performance-related testing equipment used in several European countries. The Colorado Department of Transportation (CDOT) and the FHWA Turner-Fairbank Highway Research Center (TFHRC) were selected to demonstrate this equipment.

As part of the demonstration, a device to predict the thermal cracking performance of HMA pavements was obtained. A thermal cracking device is used in Germany. Although the German device was not obtained for this demonstration, the German device as modified at Oregon State University was obtained from OEM, Inc. in Corvallis, Oregon.

## 1.1 Thermal Cracking

Thermal cracking of hot mix asphalt (HMA) pavements is the crack that is relatively perpendicular to the centerline of the pavement. Because pavements contract as the temperature decreases, stresses develop in the pavement. At very low temperatures the developed stresses exceed the pavement strength, and the pavement cracks. Thermal cracking is primarily a result of the low temperature environment.

After the thermal crack develops, water can enter the crack and cause ravelling of the joint and/or loss of base support. This significantly decreases the rideability of the pavement. In some parts of Colorado, thermal cracking is the primary distress. Thermal cracking is quantified by the frequency or spacing of the crack and the crack width.

## 1.2 Purpose

This study was designed to investigate the ability of the thermal-stress, restrained-specimen test (TSRST) to predict the ability of the HMA to resist thermal cracking. In order to evaluate the TSRST and its relationship with thermal cracking, a two-phased experiment was designed.

Phase 1 was an experiment using samples prepared in the laboratory. The purpose of Phase 1 was to evaluate several material properties that may influence thermal cracking performance. The five factors evaluated included: 1) asphalt cement stiffness, 2) asphalt cement quantity, 3) mixes with various aggregate qualities, 4) aging, and 5) the presence of hydrated lime. This information could then be used to develop project specifications for HMA materials to improve resistance to thermal cracking.

Phase 2 was an experiment using samples obtained from field pavements of known thermal cracking performance. Test results from the TSRST were compared to known field performance, and the results were intended to be used to develop specification limits that related to pavement performance.

## 1.3 Previous Studies

An excellent literature review of thermal cracking was prepared by Scherocman (*2*).

### *1.3.1 HMA Pavement Structure*

During the structural design of the HMA pavement, several decisions are typically made that will influence the thermal cracking performance. These include the type of subgrade and pavement thickness as summarized by Haas (*3*). The type of subgrade can have a substantial influence on the severity of thermal cracking. HMA pavements constructed on clay subgrades will have thermal cracks less often than those constructed on granular bases (*4*). Increasing the thickness of an HMA layer will result in less thermal cracking than thinner layers (*5*). When cracking does occur, it is generally less severe in thicker pavements.

The age of the pavement and the traffic have an influence on the thermal cracking performance. Cracking frequency increases with increasing pavement age (*4, 6*) and with higher traffic (*6*).

In some instances, overlays have been placed on HMA pavements with thermal cracking. These thermal cracks have reflected through the new overlay in a very short time, typically 1 to 2 years. If the thermal cracks are not treated in an effective rehabilitation manner prior to overlaying, the reflective cracks could give the impression of being a thermal crack. Rehabilitating thermal cracks prior to overlaying is essential to prevent the reflection of existing thermal cracks.

### *1.3.2 HMA Material*

The HMA material can be tested by using field test sections or in the laboratory. Field test sections include the St. Anne test road. The St. Anne test road indicated that the most important HMA material property influencing the thermal cracking performance is the asphalt cement stiffness (*5*).

Laboratory evaluations can be performed to identify the HMA material properties that significantly influence thermal cracking. Vinson (*7*) evaluated several test methods to predict the thermal cracking performance of HMA and recommended the use of the thermal-stress, restrained-specimen test (TSRST). Arand (*8*) of Germany uses the TSRST to evaluate the low temperature thermal cracking performance of the HMA. This device was further evaluated and developed at Oregon State University under SHRP.

Jung (*9, 10*) evaluated the factors that influenced the test results from the TSRST. They are summarized in Table 1. The most significant factor was the asphalt cement stiffness. Softer asphalt cements and more angular aggregates will improve the thermal cracking performance of an HMA.

**Table 1. Material Factors That Influence Thermal Cracking (*9, 10*).**

| | Degree of Influence on: | |
|---|---|---|
| | Fracture Temperature | Fracture Stress |
| Asphalt Cement Grade | Large | Small |
| Aging | Large | Small |
| Aggregate Type | Small | Large |
| Air Voids | Small | Large |
| Cooling rate | Large | Large |
| Correlation with SHRP Binder Tests | Excellent | |

Fabb (*11*) and Kallas (*12*) both found the asphalt cement stiffness had appreciable effects on the thermal cracking performance. Fabb (*11*) found small changes in thermal cracking performance with aggregate type, gradation, asphalt content, and cooling rate. Kallas (*12*) found that asphalt content had minimal effect on the thermal cracking performance, but the type of aggregate had an appreciable effect.

Haas (*13*) found that the frequency of thermal cracking was related to the fracture temperature based on sites of known field performance.

In summary, all of the researchers have found that the asphalt cement stiffness has a significant influence on the thermal cracking performance. Other factors have been found to be either important by some researchers but not important by others, or not important by all researchers.

# 2.0 Experimental Grid

## 2.1 Experimental Grid

This study was divided into two phases. Phase 1 is a laboratory experiment performed on laboratory prepared samples, and Phase 2 is a laboratory experiment performed on pavements of known field performance.

### *2.1.1 Phase 1: Laboratory Experiment*

The laboratory experiment used samples prepared in the laboratory. The experimental grid is shown in Table 2. Samples were tested in the thermal-stress, restrained-specimen test (TSRST) device. Four different HMA mixtures with a variety of aggregate qualities were tested at optimum asphalt content and 0.5% over optimum asphalt content. These HMA samples were tested with two different grades of asphalt cement: AC-5 and AC-20, as well as two different types of polymer modified asphalt cements: PM-ID and AC-20R. Additionally, the influence of aging and presence of hydrated lime, were investigated.

**Table 2. The Experimental Grid for Phase 1: Laboratory Experiment.**

| | AC-5 | | AC-20 | | | | PM-ID | AC-20R |
|---|---|---|---|---|---|---|---|---|
| | Opt. | OO | Opt. | OO | Short-Term Aging Only | Hydrated Lime | | |
| Mix 1 | X | X | X | | X | X | X | X |
| Mix 2 | X | X | X | X | | | | |
| Mix 3 | X | X | X | X | | | | |
| Mix 4 | X | | X | X | X | X | X | X |

Opt. - Optimum asphalt content
OO - 0.5% over optimum asphalt content
PM-ID - AASHTO Task Force 31, Type I-D (*14*)
AC-20R -AASHTO Task Force 31, Type II-B (*14*)

X - Replicate samples were tested.

The asphalt cements used in this study were tested with the SHRP binder equipment in order to determine the SHRP Performance Grade (PG) of the asphalt cement.

#### *2.1.2 Phase 2: Field Experiment*

The field experiment used samples sawn from pavements in the field. Ten sites of known field performance were identified in various parts of the state. The thermal cracking performance ranged from very poor to acceptable. Sites were also selected in some of the warmer and colder parts of the state.

Samples were sawn from the pavement at each of the ten sites. These samples were tested in the TSRST. Additionally, asphalt cement from some of the samples was extracted for testing in the bending beam rheometer (BBR). The experimental grid shown in Table 3 was performed for each of the sites.

**Table 3. The Experimental Grid for Phase 2: Field Experiment.**

| Sites | Thermal Crack | | TSRST | BBR |
|---|---|---|---|---|
| | Spacing | Width | | |
| 1-10 | X | X | X | X |

## 2.2 Description of Laboratory Tests

#### *2.2.1 Thermal-Stress, Restrained-Specimen Test*

The thermal-stress, restrained-specimen test (TSRST) is used to evaluate the resistance of the HMA to low temperature thermal cracking. The TSRST was developed at Oregon State University as part of SHRP. The TSRST is manufactured by OEM, Inc. in Corvallis, Oregon. The device is shown in Figures 1 and 2. A schematic of the sample is shown in Figure 3. The device is fully automated.

Vinson (*7*) evaluated numerous tests used to identify the low-temperature thermal cracking characteristics of HMA. Based on the evaluation, the TSRST as modified by Arand (*8*) was determined to be the best. This test has been evaluated by Jung (*9, 10*).

For this study, samples were prepared using the following procedure. The loose HMA was short-term aged for 4 hours at 135°C (270°F) and then compacted in the French plate compactor (Figure 4). Cores were taken along the length of the samples. The direction of compaction and the core are similar to the direction the thermal loading is placed on field pavements. The compacted HMA cores were then long-term aged for 120 hours (5 days) at 85°C (185°F) in a forced-draft oven. Samples tested were 50-mm (2-in.) diameter and 250-mm (10-in.) long.

After a sample is mounted in the TSRST, it is cooled at a rate of 10°C (18°F) per hour. Liquid Nitrogen is used to provide the cooling. The sample is not allowed to contract during the cooling period. The sample length is monitored with linear variable differential transformers (LVDTs) and the use of invar steel rods. Since the sample is not allowed to contract as it cools, stresses develop within it. A closed-loop system keeps the sample at a constant length. When the developed stress exceeds the strength of the sample, the sample breaks. The temperature and stress at fracture are recorded. A typical plot of the test results is shown in Figure 5.

Samples do not always break in the middle third of the sample length. When one replicate sample broke near the end and the other replicate sample broke in the middle, the fracture temperatures were typically very similar. However, it is desirable for the samples to break in the middle third. The single most important process that was believed to cause the sample to break in the middle third was its alignment. When gluing the samples, it is very important to get them aligned vertically; but this does not always ensure a break in the middle third.

The repeatability of the TSRST was studied by Jung (*9*). The coefficient of variation was 10% for the fracture temperature and 20% for the fracture strength. This was considered to be excellent and reasonable, respectively. One standard deviation, 68%, of replicate samples will have a fracture temperature within ± 2 or 3°C (± 4 or 5°F). Likewise, ± 400 to 600 kPa (± 60 to 90 psi) would be representative of fracture stresses of 68% of identical samples.

Fabb (*11*) found that fracture stress appeared to be somewhat random. Haas (*13*) found that the frequency of thermal cracking was related to the fracture temperature based on sites of known field performance.

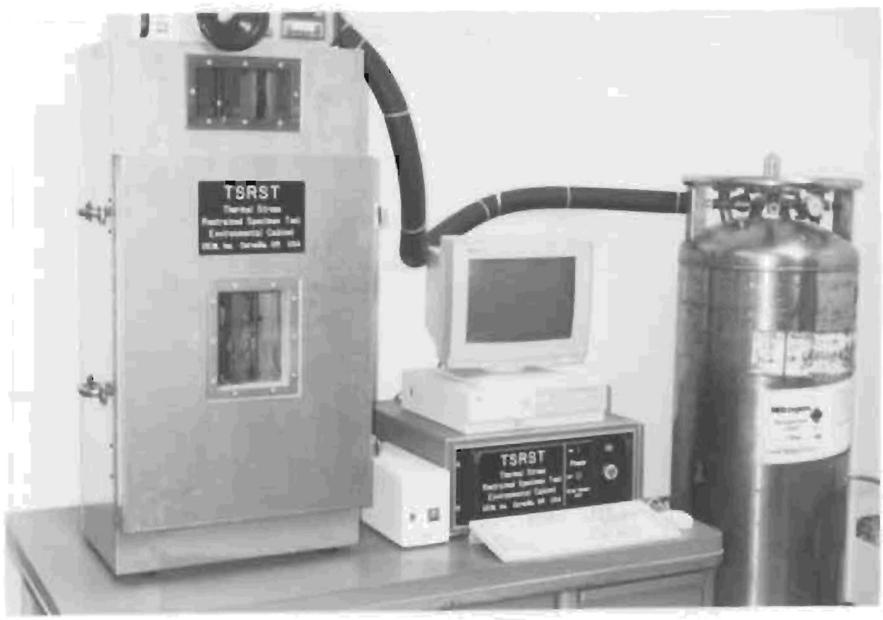


Figure 1. The TSRST Device.

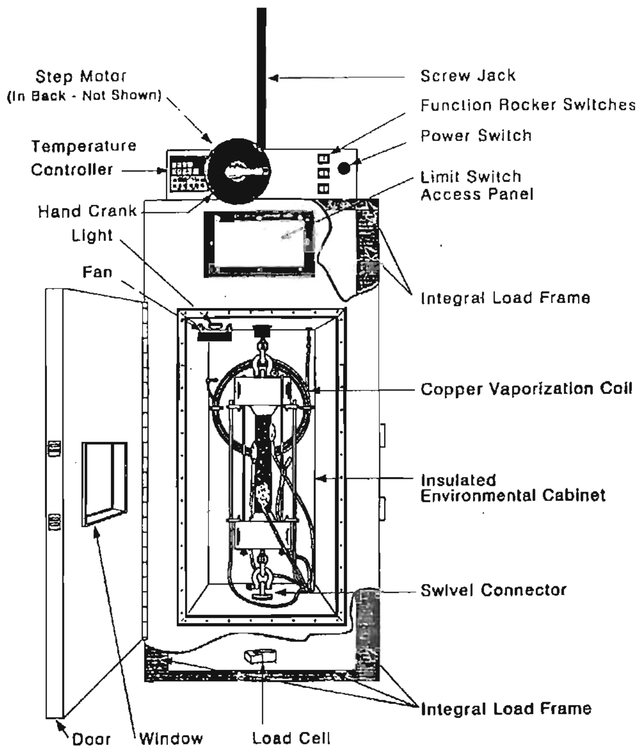


Figure 2. Schematic of the TSRST Device.

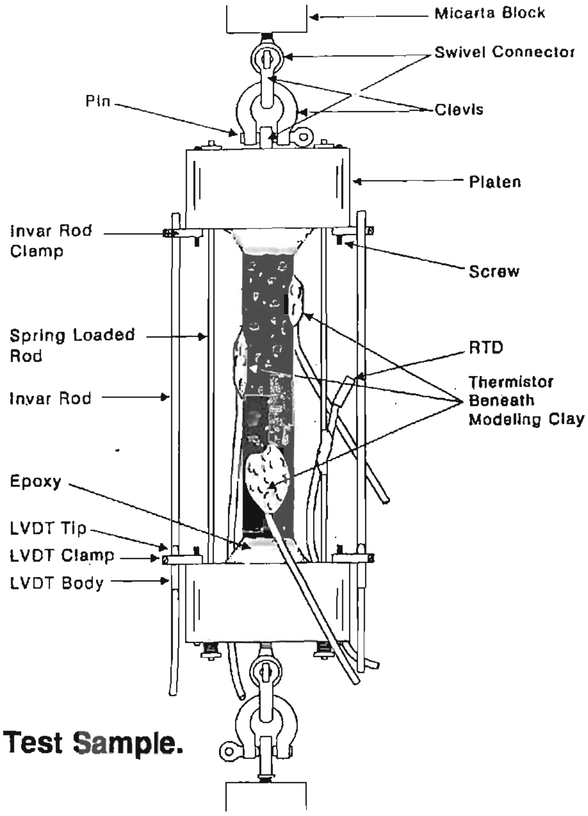


**Figure 3. Schematic of the Test Sample.**

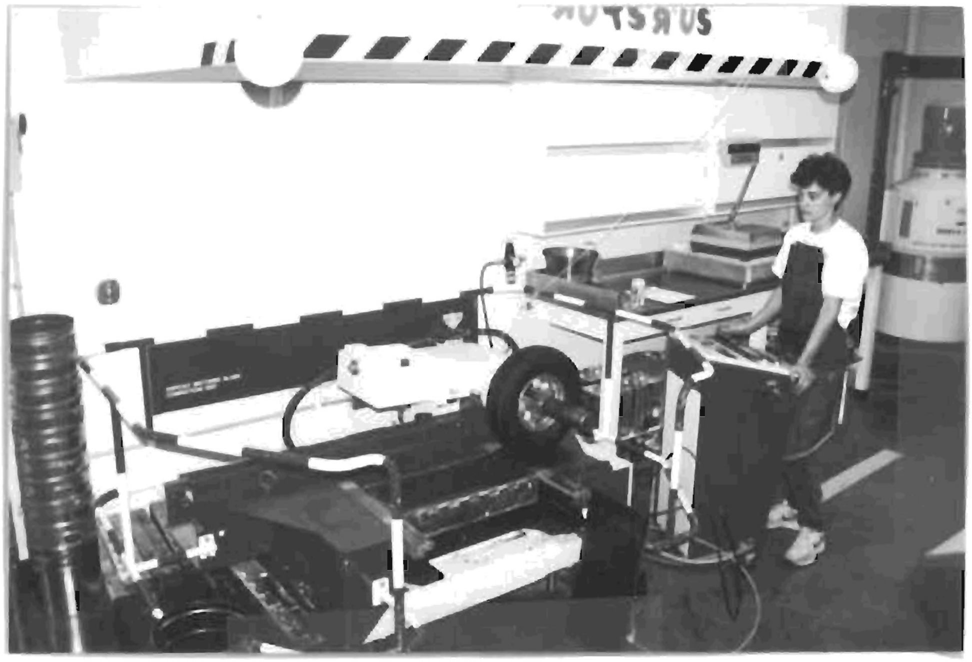

**Figure 4. The French Plate Compactor.**

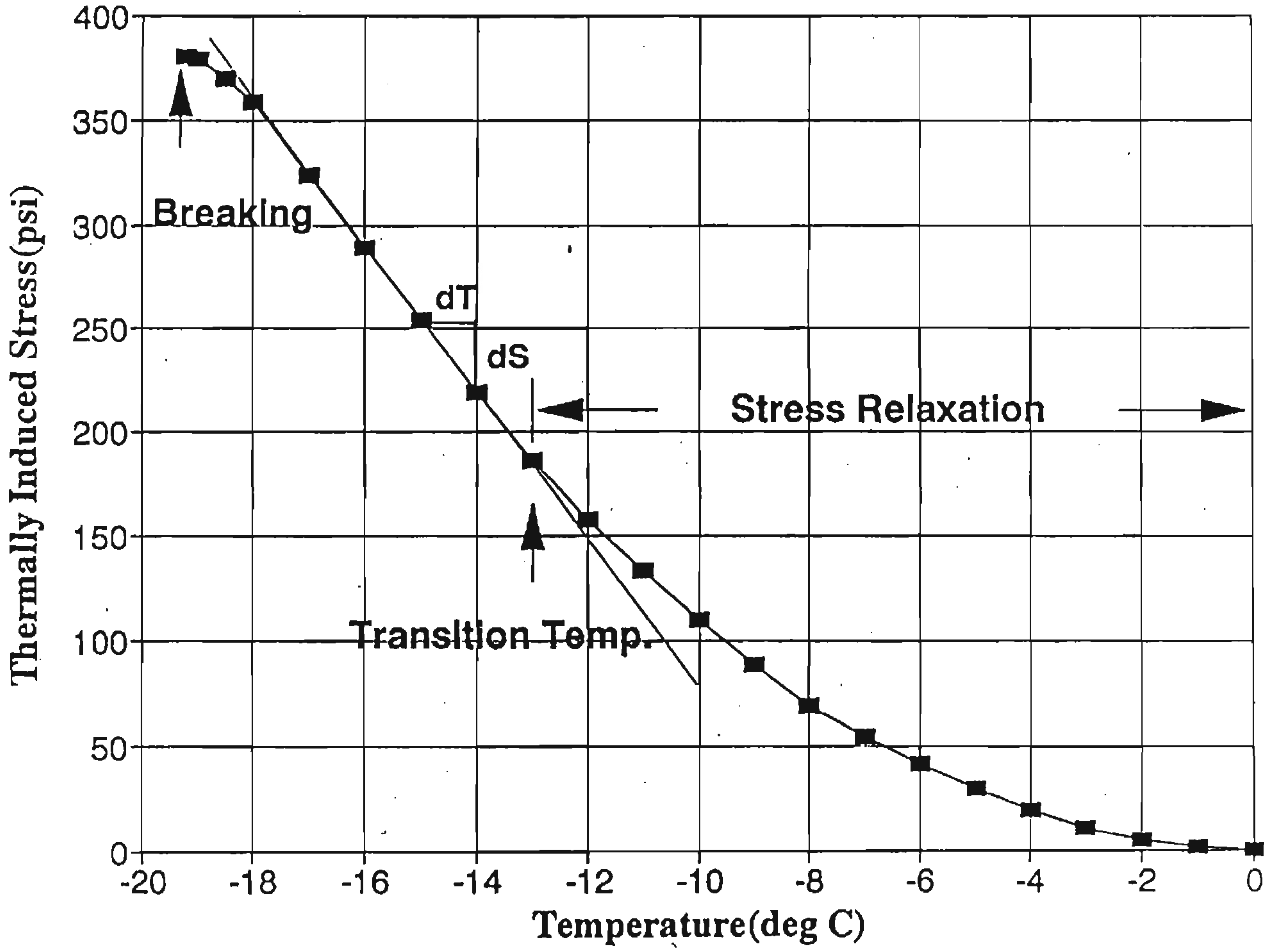


Figure 5. Typical TSRST Results.

### *2.2.2 SHRP Binder Tests*

The asphalt cement tests developed by SHRP were performed. These tests are used to identify the resistance of the asphalt cement's contribution to rutting, thermal cracking, and fatigue of the HMA.

A full series of tests were performed to determine the SHRP Performance Grade (PG) of each asphalt cement. The testing devices were the dynamic shear rheometer (DSR) and bending beam rheometer (BBR). The tests were performed on asphalt cements that were unaged (tank), rolling thin film oven test (RTFOT) aged (AASHTO T 240), and pressure aging vessel (PAV) aged.

The DSR is used to measure the ability of the asphalt cement to resist permanent deformation at high temperatures and resist fatigue cracking at moderate temperatures.

The BBR is used to measure the ability of the asphalt cement to resist thermal cracking at low temperatures. The test results from the BBR were most useful for this research study. The results from the BBR are controlled by one of two specifications. The <u>first</u> specification is the temperature at which a stiffness of 300.0 MPa is achieved. Temperatures that cause the binder to be stiffer than 300.0 MPa will likely be temperatures that cause thermal cracking. The <u>second</u> specification is the temperature at which the slope (m-value) is equal to 0.300. Temperatures that cause the binder to have slopes less than 0.300 will likely be temperatures that cause thermal cracking. The warmest of the two temperatures (from either stiffness or slope) then controls the low temperature performance grading of the binder.

# 3.0 Phase 1: Laboratory Experiment

## 3.1 Material Properties

### *3.1.1 Asphalt Cement Properties*

Each of the asphalt cements was graded with the SHRP binder equipment. The SHRP Performance Grade (PG) is shown in Table 4. The detailed SHRP results are shown in Appendix A. The first number of the SHRP PG represents the highest 7-day average pavement temperature for which the binder should be used. For example, the AC-20 should be used on pavements which will have a highest 7-day average pavement temperature of less than 64°C. The second number represents the lowest pavement temperature for which the binder should be used; it is the temperature from the bending beam rheometer (minus 10°C) that gives a slope of 0.300. For example, an AC-20 should be used for pavements which will have a coldest temperature that is warmer than -22°C. For this phase of the thermal cracking study, the low temperature grading was always controlled by the slope (m-value) from the BBR.

**Table 4. SHRP Performance Grade (PG) of the Asphalt Cements.**

| | SHRP PG | BBR (°C)* at m-value of 0.300 |
|---|---|---|
| AC-5 | 52-28 | -18.0 |
| AC-20 | 64-22 | -12.3 |
| PM-ID | 76-28 | 19.0 |
| AC-20R | 64-22 | -14.9 |

PM-ID - AASHTO Task Force 31, Type I-D (*14*)
AC-20R -AASHTO Task Force 31, Type II-B (*14*)
* To adjust for loading rate, subtract 10°C (*15*).

### *3.1.2 Aggregate Properties*

Aggregates were selected to represent a wide variety of thermal cracking performance that can exist in Colorado. Aggregates thought to have good and poor thermal cracking performance characteristics were obtained. Both coarse and fine aggregates were selected. Results of tests performed on the aggregates are shown in Table 5. All four combinations of aggregates were tested: for example, coarse aggregates having good thermal cracking performance and fine aggregates having poor thermal cracking performance (Mix 2) as shown in Table 6.

Acceptable methylene blue values are less than or equal to 10 mg/g. Acceptable fine aggregate angularity test results are greater than or equal to 45.0.

**Table 5. Aggregate Properties.**

| Test / Method | Coarse Aggregate | | Fine Aggregate | |
|---|---|---|---|---|
| | Good Performer | Poor Performer | Good Performer | Poor Performer |
| Sand Equivalent AASHTO T 176 | NA | NA | 65 | 34 |
| Plasticity Index AASHTO T 90 | Not Plastic | Not Plastic | Not Plastic | Not Plastic |
| Methylene Blue Value (mg/g) ISSA, Technical Bulletin 145 | NA | NA | 8.1 | 20+ |
| Two or More Fract. Faces (%) .CP-45 | 96 | 82 | NA | NA |
| Fine Aggregate Angularity (%) AASHTO TP 33 | NA | NA | 47.8 | 41.2 |
| Water Absorption (%) AASHTO T 84 or 85 | 0.8 | 2.6 | 1.2 | 1.0 |

NA - Not Applicable

**Table 6. Combination of Aggregates Tested.**

| | | Coarse Aggregate | |
|---|---|---|---|
| | | Good | Poor |
| Fine Aggregate | Good | Mix 1 | Mix 3 |
| | Poor | Mix 2 | Mix 4 |

### *3.1.3 HMA Properties*

The optimum asphalt content of the mixes tested in this study are shown in Table 7. Optimum asphalt contents were determined using the Texas gyratory compactor (ASTM D 4013) with a 520 kPa (75 psi) end-point stress. The voids in the mineral aggregate (VMA) are also shown in Table 7. Each mix had a minimum VMA requirement of 14.0 for the 12.5-mm (1/2-in.) nominal maximum aggregate size.

**Table 7. Summary of Optimum Asphalt Content for the HMAs.**

| Mix | Optimum AC (%) | Air Voids (%) | VMA (%) | Aggregate Quality | |
|---|---|---|---|---|---|
| | | | | Coarse | Fine |
| 1 | 6.0 | 4.0 | 15.5 | Good | Good |
| 2 | 6.8 | 4.0 | 17.0 | Good | Poor |
| 3 | 6.2 | 4.0 | 15.7 | Poor | Good |
| 4 | 6.3 | 4.0 | 16.5 | Poor | Poor |

Mixes 2 and 4 each had very poor fine aggregate that had a high clay content. The high VMA for these two mixes was attributed to the large amount of asphalt demand needed by the fine aggregate.

Gradations of the four mixes are not the same. The gradations are plotted in Appendix B.

## 3.2 Test Results and Discussion

Two replicate samples were always tested, and the averaged temperatures and stresses at fracture are summarized in Tables 8 and 9, respectively.

**Table 8. Summary of Temperature (°C) at Fracture from the TSRST.**

| | AC-5 | | AC-20 | | | | PM-ID | AC-20R |
|---|---|---|---|---|---|---|---|---|
| | Opt. | OO | Opt. | OO | Short-Term Aging Only | Hydrated Lime | | |
| Mix 1 | -29 | -31 | -24 | | -26 | -27 | -34 | -28 |
| Mix 2 | -30 | -30 | -25 | -24 | | | | |
| Mix 3 | -30 | -29 | -25 | -24 | | | | |
| Mix 4 | -29 | | -25 | -22 | -27 | -28 | -31 | -28 |

Opt. - Optimum asphalt content
OO - 0.5% over optimum asphalt content
PM-ID and AC-20R - polymer modified asphalt cements

**Table 9. Summary of Stress (kPa) at Fracture from the TSRST.**

| | AC-5 | | AC-20 | | | | PM-ID | AC-20R |
|---|---|---|---|---|---|---|---|---|
| | Opt. | OO | Opt. | OO | Short-Term Aging Only | Hydrated Lime | | |
| Mix 1 | 2510 | 2610 | 2380 | | 2600 | 2660 | 3470 | 2810 |
| Mix 2 | 2560 | 2340 | 2420 | 2210 | | | | |
| Mix 3 | 1970 | 2170 | 2400 | 2240 | | | | |
| Mix 4 | 2010 | | 1940 | 1990 | 2240 | 2120 | 2890 | 2680 |

Opt. - Optimum asphalt content
OO - 0.5% over optimum asphalt content
PM-ID and AC-20R - polymer modified asphalt cements

The average temperature at fracture was compared for the variables tested in this study. The comparisons are summarized in Table 10. The differences reported in Table 10 are the fracture temperature of the first variable minus the fracture temperature of the second variable. A negative difference indicates the first variable had a lower or colder fracture temperature than the second variable. A positive difference indicates the first variable had a higher or warmer fracture temperature than the second variable.

**Table 10. Comparison of Fracture Temperature (°C) with Material Properties.**

| Comparison | n | Diff. | S.D. |
|---|---|---|---|
| Asphalt: | | | |
| AC-5 to AC-20 | 6 | -5.0 | 0.6 |
| PM-ID to AC 5 | 2 | -3.5 | NA |
| PM-ID to AC-20 | 2 | -8.0 | NA |
| AC-20R to AC-5 | 2 | 1.0 | NA |
| AC-20R to AC-20 | 2 | -3.5 | NA |
| Opt. to OO | 6 | -0.7 | 1.6 |
| Aggregate: | | | |
| Mix 1 to Mix 2 | 3 | +0.3 | 1.2 |
| Mix 1 to Mix 3 | 3 | 0.0 | 1.7 |
| Mix 1 to Mix 4 | 6 | 0.0 | 1.5 |
| Mix 2 to Mix 3 | 4 | -0.3 | 0.5 |
| Mix 2 to Mix 4 | 3 | -1.0 | 1.0 |
| Mix 3 to Mix 4 | 3 | -1.0 | 1.0 |
| Others: | | | |
| Lime to No Lime | 2 | -3.0 | NA |
| Aging: Short to Long | 2 | -2.0 | NA |

NA - Not Applicable
S.D. - standard deviation

### *3.2.1 Influence of Asphalt Cement Stiffness*

The asphalt cement stiffness had a large effect on the fracture temperature. The AC-5 asphalt cement had a 5°C colder fracture temperature than the AC-20 (Table 10): approximately a 20% difference. The fracture stress between the AC-5 and AC-20 was not significantly changed (Table 9).

The use of polymer modified asphalt cements significantly improved the low temperature thermal cracking performance. The PM-ID had an 8°C lower fracture temperature than the AC-20, approximately a 30% difference, and a 3.5°C lower fracture temperature than the AC-5, approximately a 12% difference. Additionally, the fracture stress for the PM-ID increased 1000 kPa (145 psi), approximately 50%, over the fracture stresses of the unmodified asphalt cements (Table 9).

The AC-20R had a 3.5°C lower fracture temperature than the AC-20 and a 1°C higher fracture temperature than the AC-5 (Table 10). The fracture stress for the AC-20R increased 500 kPa (73 psi), approximately 25%, over the fracture stresses of the unmodified asphalt cements (Table 9).

The polymer modified asphalt cements decreased the fracture temperature and increased the fracture stress substantially. The AC-20R did not perform as well as the PM-ID.

### *3.2.2 Influence of Asphalt Content*

By increasing the asphalt content 0.5%, the fracture temperature and fracture stresses were not changed. The quantity of asphalt cement, within reasonable proximity to the optimum asphalt content (0.5%), did not influence the test results from the TSRST.

### *3.2.3 Influence of Aggregate Quality*

A wide variety of aggregate quality was used in this experiment. There was virtually no difference in the fracture temperature between the various aggregates (Table 10). Aggregate quality had little influence, less than or equal to 1°C, on fracture temperature in this study. Fracture strength increased 500 kPa (73 psi), approximately 20%. The mix with better aggregate (Mix 1) had higher strengths than the mix with poorer aggregates (Mix 4).

### *3.2.4 Influence of Aging*

Aging had an influence on the fracture temperature and fracture stress. Samples that were only short-term aged had a fracture temperature about 2°C colder than samples that were short-term and long-term aged (Table 10). Additional aging increased the fracture temperature.

The fracture stress for samples that were short-term aged was greater than the fracture stress for samples that were short-term and long-term aged. Samples that were only short-term aged had fracture stresses 300 kPa (44 psi) higher, approximately 15%, than samples that were long-term aged (Table 9).

### *3.2.5 Influence of Hydrated Lime*

The CDOT currently uses hydrated lime in approximately 90% of the HMA for anti-stripping purposes. The use of hydrated lime lowered the fracture temperature about 3°C compared to samples without hydrated lime (Table 10). When using hydrated lime, the fracture stress increased about 250 kPa (36 psi), approximately 10% (Table 9).

### *3.2.6 Summary*

A summary of the influence of all of the variables tested in this study is shown in Table 11. The summary is for influence of both fracture temperature and stress. The percentages are for the change from the first variable to the second variable. For example, switching from AC-20 to AC-5 would cause the fracture temperature to decrease approximately 20% but cause no change to the fracture strength. A decrease in fracture temperature and an increase in fracture strength are considered beneficial. The use of polymer modified asphalt cements had the only significant change in <u>both</u> fracture temperature and stress.

**Table 11. Summary of Influence of Several Variables on Thermal Cracking Performance.**

| | Fracture Temperature | Fracture Stress |
|---|---|---|
| AC-20 to AC-5 | - 20% | 0% |
| AC-20 to PM-ID | - 30% | + 50% |
| Poor to Good Aggregate | 0% | + 20% |
| No Lime to Lime | - 10% | + 10% |
| Aging: Long to Short | - 10% | + 15% |

The variables shown in Table 11 have a very similar influence to thermal cracking as those reported in Table 1 by Jung (*9, 10*). A significant change was considered to be larger than 10%

for temperature and larger than 20% for stress based on the data presented in Section 2.2.1 from Jung (*9*).

### *3.2.7 Comparison of Mixture and Binder Test Results*

The fracture temperature of the HMA measured by the TSRST was compared to the temperature at which the asphalt cement had a slope of 0.300 in the BBR. The comparison is shown in Table 12. For the HMA samples, all of the different aggregates were grouped together. The HMA and asphalt cement comparisons are very uniform. For samples prepared and aged in the laboratory, the BBR results on the binder gave approximately 2°C to 3°C <u>warmer</u> temperatures than the TSRST results on the mix.

**Table 12. Comparison of TSRST Fracture Temperature and the BBR.**

| | TSRST | | | BBR (°C)* at m-value of 0.300 | Difference Between HMA and Asphalt Cement |
|---|---|---|---|---|---|
| | n | Avg. | S.D. | | |
| AC-5 | 14 | -29.8 | 1.2 | -28.0 | 1.8 |
| AC-20 | 14 | -24.0 | 1.3 | -22.3 | 1.7 |
| PM-ID | 8 | -32.3 | 2.0 | -29.0 | 3.3 |
| AC-20R | 8 | -27.6 | 2.6 | -24.9 | 2.7 |

* To account for the loading rate in the laboratory, the temperature has been adjusted by 10°C to give the field temperature (*15*).

# 4.0 Phase 2: Field Experiment

## 4.1 Site Selection

Sites of known thermal cracking performance were selected throughout Colorado. The site locations and pavement conditions are summarized in Table 13.

**Table 13. Thermal Cracking Sites.**

| Site | Region | Highway | Location | Thermal Cracking | | | Low Temp.* °C (°F) |
|---|---|---|---|---|---|---|---|
| | | | | Width mm (in) | Spacing m (ft) | Severity | |
| 1 | 4 | SH-63 M.P. 2 | Anton | 12 (½) | 5-9 (15-30) | High | -26 (-15) |
| 2 | 4 | SH-71 M.P. 141 | Last Chance | 6 (¼) | 24-27 (80-90) | Low | -26 (-15) |
| 3 | 4 | US-385 M.P. 284 | Holyoke | 12 (½) | 5-6 (15-20) | High | -26 (-15) |
| 4 | 5 | SH-24 M.P. 194 | Buena Vista | 19-25 (¾-1) | 24-46 (80-150) | Med | -27 (-17) |
| 5 | 5 | SH-17 M.P. 119 | Moffat | 12 (½) | 15-21 (50-70) | Med | -27 (-17) |
| 6 | 5 | SH-149 M.P. 26 | Creede | 19-38 (¾-1½) | 2-3 (5-10) | High | -32 (-26) |
| 7 | 2 | US-50 M.P. 298 | Pueblo | 19-25 (¾-1) | 6-8 (20-25) | High | -24 (-11) |
| 8 | 2 | US-50 M.P. 348 | Manzanola | 12-19 (½-¾) | 6 (20) | High | -26 (-15) |
| 9 | 2 | SH-96 M.P. 84 | Boone | 6 (¼) | 8 (25) | Med | -26 (-15) |
| 10 | 2 | SH-69 M.P. 20 | Farasita | 19-25 (¾-1) | 6-8 (20-25) | High | -26 (-15) |

* - Average of lowest pavement temperatures (50% reliability)

The severity of the thermal crack was defined as low, medium or high. Low severity cracks were less than 6 mm (0.25 in.) and had no raveling. Medium severity cracks have widths greater than 6 mm with no raveling or less than 6 mm with raveling. High severity cracks have widths greater than 6 mm and raveling. The crack widths reported in Table 13 were measured in the summer.

The low temperature environment of the pavements were determined from the average of the lowest pavement temperatures. If this temperature were a design temperature, it would represent a 50% reliability. In other words, there would be a 50% chance the temperature in a given year would be lower than the design temperature. These temperatures were obtained from the SHRP weather data base.

No sample could be retrieved from Site 5, because it was severely stripped. Therefore, no testing was performed on this site. Site 5 was included in Table 13 to serve as an example of the confounding variables that exist with the field sites. The core from Site 4 was very rough on the sides. It was also very weak from stripping. Site 10 had no test results because problems developed with the load cell on the TSRST, so the samples were not tested properly.

## 4.2 Test Results and Discussion

The test results from the TSRST and the bending beam rheometer (BBR) are shown in Table 14. Also shown is the ranking of the known field performance from best (1) to worst (10). It is important to mention that the asphalt cement tested in the BBR was extracted from the field slabs and recovered using the Abson method (AASHTO T 170). It is well known that recovering the asphalt cement may alter the asphalt cement. The BBR test results are shown in Appendix C.

### *4.2.1 Comparison of Mixture and Binder Test Results.*

The binder properties measured with the bending beam rheometer (BBR) were correlated with the mixture results measured from the TSRST and the results are shown in Figure 6.

The BBR results can be controlled by the minimum slope (m-value) specification of 0.300 or the maximum stiffness specification of 300.0 MPa. In all cases, the temperatures were controlled by the m-value. The temperature from the BBR minus 10°C is equivalent to the lowest pavement

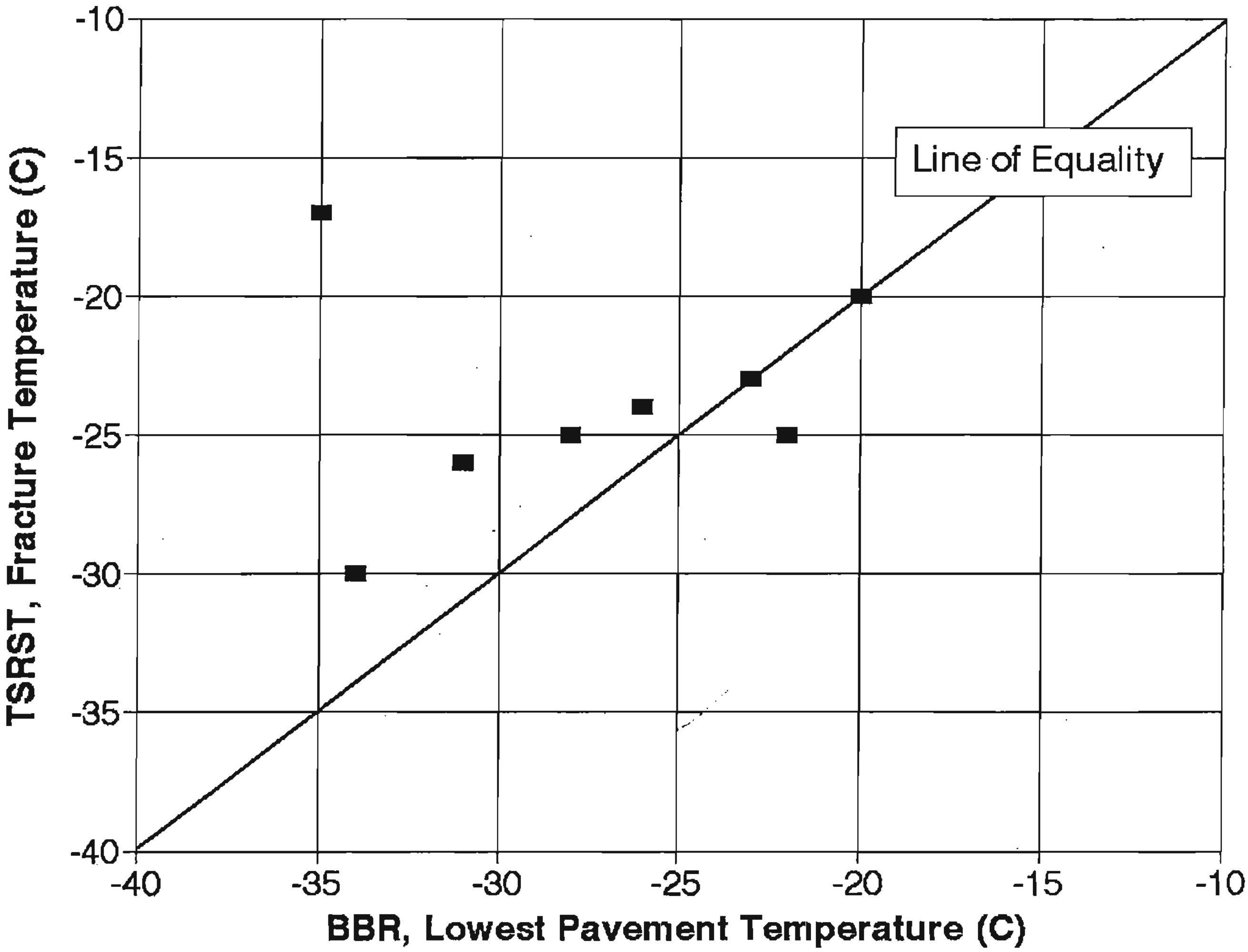


Figure 6. Correlation Between the TSRST and BBR Temperatures.

temperature that will provide performance. The temperatures from the BBR shown in Table 14 are the actual test results, and the temperatures shown in Figure 6 were corrected to the pavement temperature.

**Table 14. Summary of TSRST and BBR Results on Field Pavements.**

| Site | TSRST Results at Fracture | | Low Temperature in the Field (°C) | | BBR °C @ m-value of 0.300 | Rank of Field Thermal Cracking | |
|---|---|---|---|---|---|---|---|
| | Temp. (°C) | Stress (kPa) | Reliability | | | Based on Width | Based on Spacing |
| | | | 50% | 98% | | | |
| 1 | -23 | 1890 | -26 | -32 | -13 | 3 | 8 |
| 2 | -24 | 2290 | -26 | -32 | -16 | 1 | 2 |
| 3 | -25 | 1980 | -26 | -34 | -12 | 3 | 9 |
| 4 | -30 | 3490 | -27 | -33 | -24 | 7 | 1 |
| 5 | NT | NT | -27 | -35 | NT | 3 | 3 |
| 6 | -17 | 1190 | -32 | -38 | -25 | 10 | 10 |
| 7 | -25 | 2050 | -24 | -32 | -18 | 7 | 5 |
| 8 | -26 | 2980 | -26 | -34 | -21 | 6 | 7 |
| 9 | -20 | 1750 | -26 | -34 | -10 | 1 | 4 |
| 10 | NT | NT | -26 | -34 | -11 | 7 | 5 |

NT - Not Tested.

Data that plots on the line of equality in Figure 6 indicates that the BBR and TSRST temperatures are the same. For seven of the eight sites, there appears to be a reasonably close relationship. One of the sites (Site 6) is not very close. Although the TSRST indicated there would be poor performance and the BBR indicated there would be good performance, the field temperature exceeded both. Therefore, it is not possible to tell if the BBR or TSRST was more accurate.

For field-aged samples, the BBR results on the binder gave approximately 2°C cooler temperatures than the TSRST results on the mix. The range was approximately 0°C to 5°C cooler.

Site 3 provided an interesting difference between the test results from the TSRST and the BBR. From the BBR, the lowest test temperature based on the m-value was -12°C and the lowest test temperature based on the stiffness was -27°C; this is a large disparity. For the other sites, the two temperatures from the BBR were much closer. By using the minimum temperature as determined by the stiffness value from the BBR, Site 3 would fall more in line with the ranking determined by the temperature at fracture from the TSRST.

*4.2.2 Correlation of Laboratory Test Results and Field Performance.*

Test results from the TSRST and the BBR were correlated with the actual field performance. The sites of known field performance were ranked in order from best to worst performance. The performance ranking was determined based on crack spacing (Table 15) and crack width (Table 16). The further apart the spacing, the better the ranking. The narrower the crack, he better the ranking.

**Table 15. Ranking Based on Crack Spacing.**

| Rank | Site Numbers | | | |
|---|---|---|---|---|
| | Field | BBR | TSRST (Temp) | TSRST (Strgth) |
| 1 | 4 | 6 | 4 | 4 |
| 2 | 2 | 4 | 8 | 8 |
| 3 | 5 | 8 | 3,7 | 2 |
| 4 | 9 | 7 | 3,7 | 7 |
| 5 | 7, 10 | 2 | 2 | 3 |
| 6 | 7, 10 | 1 | 1 | 1 |
| 7 | 8 | 3 | 9 | 9 |
| 8 | 1 | 10 | 6 | 6 |
| 9 | 3 | 9 | ** | ** |
| 10 | 6 | * | ** | ** |

* Site 5 was not tested
** Sites 5 and 10 were not tested

**Table 16. Ranking Based on Crack Width.**

| Rank | Site Numbers | | | |
|---|---|---|---|---|
| | Field | BBR | TSRST (Temp) | TSRST (Strgth) |
| 1 | 2, 9 | 6 | 4 | 4 |
| 2 | 2, 9 | 4 | 8 | 8 |
| 3 | 1, 3, 5 | 8 | 3, 7 | 2 |
| 4 | 1, 3, 5 | 7 | 3,7 | 7 |
| 5 | 1, 3, 5 | 2 | 2 | 3 |
| 6 | 8 | 1 | 1 | 1 |
| 7 | 4, 7, 10 | 3 | 9 | 9 |
| 8 | 4, 7, 10 | 10 | 6 | 6 |
| 9 | 4, 7, 10 | 9 | ** | ** |
| 10 | 6 | * | ** | ** |

* Site 5 was not tested
** Sites 5 and 10 were not tested

Rankings based upon crack spacing and crack width were not consistently similar to the known field performance. The most promising ranking, based on regression, was between the field performance as measured by crack spacing with the fracture strength measured by the TSRST. The coefficient of determination, $r^2$, was 0.49. The plot is shown in Figure 7.

It is not surprising that the correlation with the field performance is so varied. There were numerous variables that were difficult to control for this field study. Thermal cracking is a function of subgrade type and pavement thickness which are not accounted for by material properties. The process of recovering the asphalt cement using the Abson method will influence the properties of the binder. The field performance of thermal cracking could have had some influence from reflective cracking or moisture damage. Actual field temperatures during the life of the pavement may not have matched the statistically predicted trend.

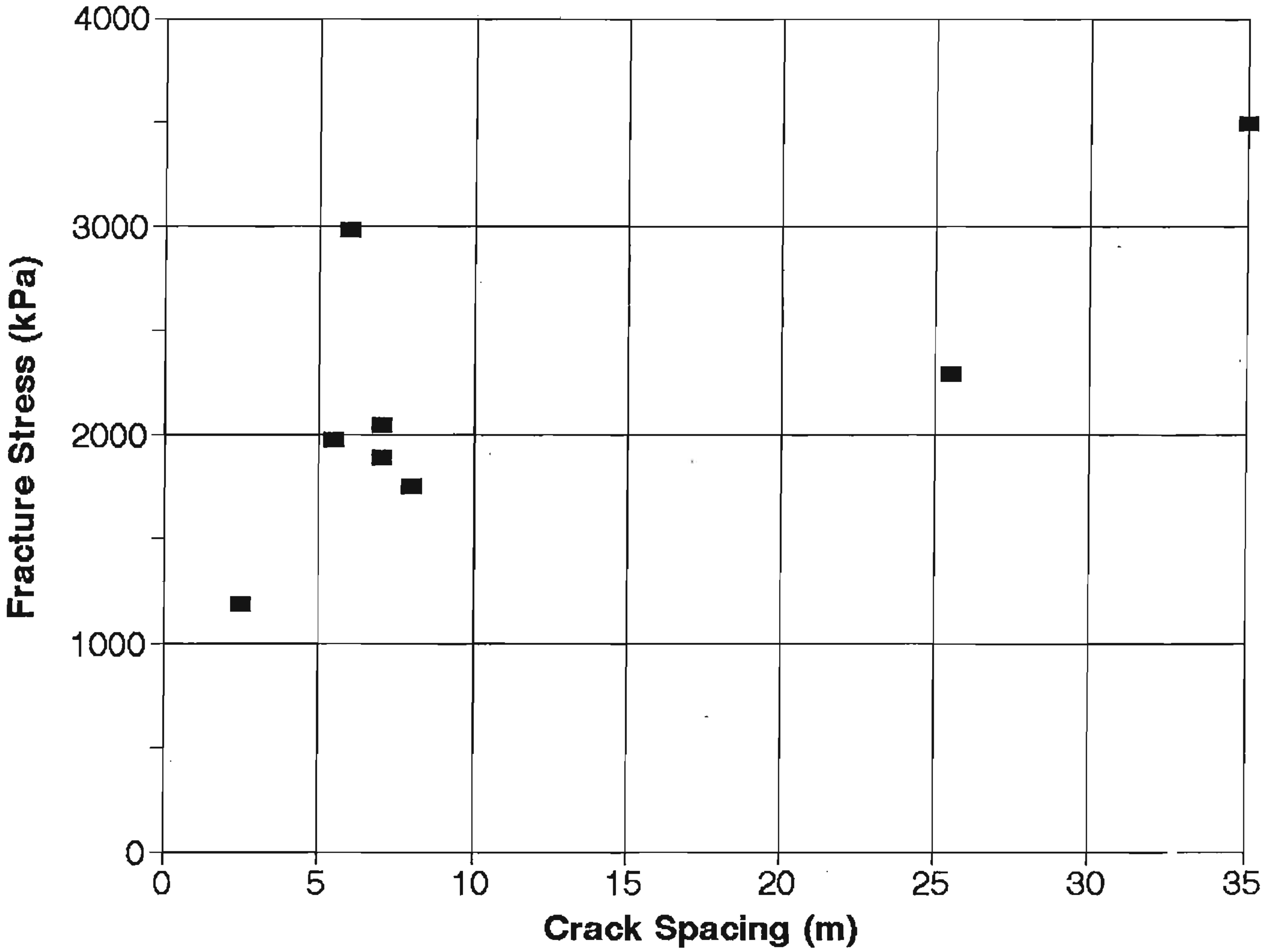


Figure 7. Correlation of Crack Spacing with Fracture Strength from the TSRST.

# 5.0 Conclusions

The following conclusions are limited to the materials tested in this study.

## 5.1 Phase 1: Laboratory Experiment

A laboratory study was conducted to determine the variability of test results as an influence of 1) asphalt cement stiffness, 2) asphalt cement quantity, 3) mixes with various aggregate qualities, 4) aging, and 5) the presence of hydrated lime.

1) Thermal cracking performance of HMA as measured by the fracture temperature is more sensitive to the asphalt cement stiffness than any other variable investigated. The fracture temperature is also sensitive to the degree of aging and presence of hydrated lime. HMAs have lower fracture temperatures when the HMA has softer asphalt cement, shorter aging, and hydrated lime.

2) Thermal cracking performance of HMA as measured by the fracture stress is more sensitive to the presence of a polymer modifier than any other variable investigated. The fracture stress is also sensitive to the aggregate quality, degree of aging, and presence of hydrated lime. HMAs have higher fracture stresses when the HMA has polymer modifiers, good quality aggregate, shorter aging, and hydrated lime.

3) Thermal cracking performance of HMA as measured by the fracture temperature is not sensitive to the asphalt content (within 0.5% over optimum) or aggregate quality. The fracture stress was not sensitive to the asphalt content.

4) Polymer modifiers were the only variable that significantly improved both the fracture temperature and stress of the HMA.

5) Correlation of the bending beam rheometer (BBR) and the thermal-stress restrained-specimen test (TSRST) was very good. When the binder and mix were both aged in the laboratory with their respective procedures, the BBR results on the binder gave approximately 2°C warmer temperatures than the TSRST results on the mix. The BBR should provide reasonable approximations of the low-temperature thermal cracking performance of the HMA.

## 5.2 Phase 2: Field Experiment

1) Test results on the binder from the BBR and on the mixture from the TSRST gave similar results. When the binder and mix were both aged in the field, the BBR results on the binder gave approximately 2°C cooler temperatures than the TSRST results on the mix.

2) Correlation of the test results to the known field performance was not very good. The best correlation was with the fracture strength measured by the TSRST and the m-value from the BBR.

It is not surprising that the correlation with the field performance is so varied. There were numerous variables that were difficult to control for this field study. Thermal cracking is a function of subgrade type and pavement thickness which are not accounted for when binder or mix material properties are measured. Additionally, the process of recovering the asphalt cement using the Abson method will influence the properties of the binder. The field performance of thermal cracking could have had some influence from reflective cracking or moisture damage. Actual field temperatures during the life of the pavement may not have matched the statistically predicted trend.

# 6.0 Recommendations

Pavement Management. When designing a project to resist thermal cracking, it is necessary to begin with the pavement structure. If the project is an overlay, existing thermal cracks must be treated, preferably with cold-in-place or hot-in-place recycling. The overlay should be thick, a minimum of 50 mm (2 inches). Thermal cracking that develops in thicker overlays is less severe than that in thinner overlays.

Material Specifications. After the pavement structure is addressed properly, the material properties of the HMA can increase the resistance to thermal cracking. Primarily, softer asphalt cements will be the biggest factor that improves the resistance to thermal cracking. If softer asphalt cements create concerns about the high temperature rutting, then polymer modified asphalt cements should be used.

The bending beam rheometer, a SHRP binder test, appears to provide very similar results to the thermal-stress, restrained-specimen test for predicting the thermal cracking of the HMA. Additionally, the bending beam rheometer is fast and easy to perform. The bending beam rheometer test on the asphalt cement should be used as a specification to improve thermal cracking resistance of the HMA.

Tests on the HMA itself will be beneficial when trying to design or investigate pavements undergoing reconstruction or for high volume roadways. Additionally, when trying to predict the quantity of thermal cracking by using performance models, mix tests will be very useful. The TSRST data could be used to predict actual pavement performance from performance models in cases of complete reconstruction or special high volume pavements.

## 7.0 Future Research

The use of wearing surfaces might improve the resistance of the HMA pavement to thermal cracking. Wearing surfaces include the plant-mixed seal coat (PMSC) and open graded friction coarse (OGFC). Additionally, stone mastic asphalt (SMA) pavements may resist thermal cracking better than our standard dense graded HMAs. The thermal cracking performance of PMSC, OGFC, and SMA should be investigated.

Results from this study indicated that polymers significantly improved the fracture stress using the TSRST. The direct tension test on binders should be used to investigate the fracture stress of the binder, particularly with polymer modified asphalt cement.

The study could be expanded to isolate pavement thickness, pavement age, subgrade type, etc. in order to get a better correlation between laboratory tests and field performance. This would probably be an unmanageable study. Therefore, evaluating the BBR results for future field projects should be acceptable.

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## Appendix A:

## SHRP Binder Test Results from the Laboratory Experiment.

| Aging | Test | Test Temp. °C | Units of Results | Binder Type AC-5 | AC-20 | |
|---|---|---|---|---|---|---|
| Tank | Sp.Gr. | 25 | | | | |
| | Flash | | °C | 282 | 293 | |
| | Ab.Vis. | 60 | poises | | | |
| | Pen | 25 | dmm | | | |
| | DSR | 48 | kPa | 3.39 | | |
| | | 52 | kPa | 1.56 | 6.90 | |
| | | 58 | kPa | 0.73 | 3.03 | |
| | | 64 | kPa | | 1.30 | |
| | | 70 | kPa | | 0.60 | |
| RTFOT | LOH | 163 | % | 0.14 | 0.06 | |
| | DSR | 46 | kPa | 7.75 | | |
| | | 52 | kPa | 3.36 | | |
| | | 58 | kPa | 1.48 | 6.57 | |
| | | 64 | kPa | | 2.80 | |
| | | 70 | kPa | | 1.24 | |
| PAV | DSR | 25 | kPa | 1,140 | 2,550 | |
| | | 22 | kPa | 1,760 | 3,630 | |
| | | 19 | kPa | 2,610 | 5,040 | |
| | | 16 | kPa | 3,750 | | |
| | | 13 | kPa | 5,430 | | |
| | BBR Stiffness (S) | -12 | MPa | 100 | | |
| | | -18 | MPa | 389 | 222 | |
| | | -24 | MPa | 433 | 404 | |
| | BBR Slope (m) | -12 | | 0.377 | 0.302 | |
| | | -18 | | 0.296 | 0.261 | |
| | | -24 | | 0.281 | | |

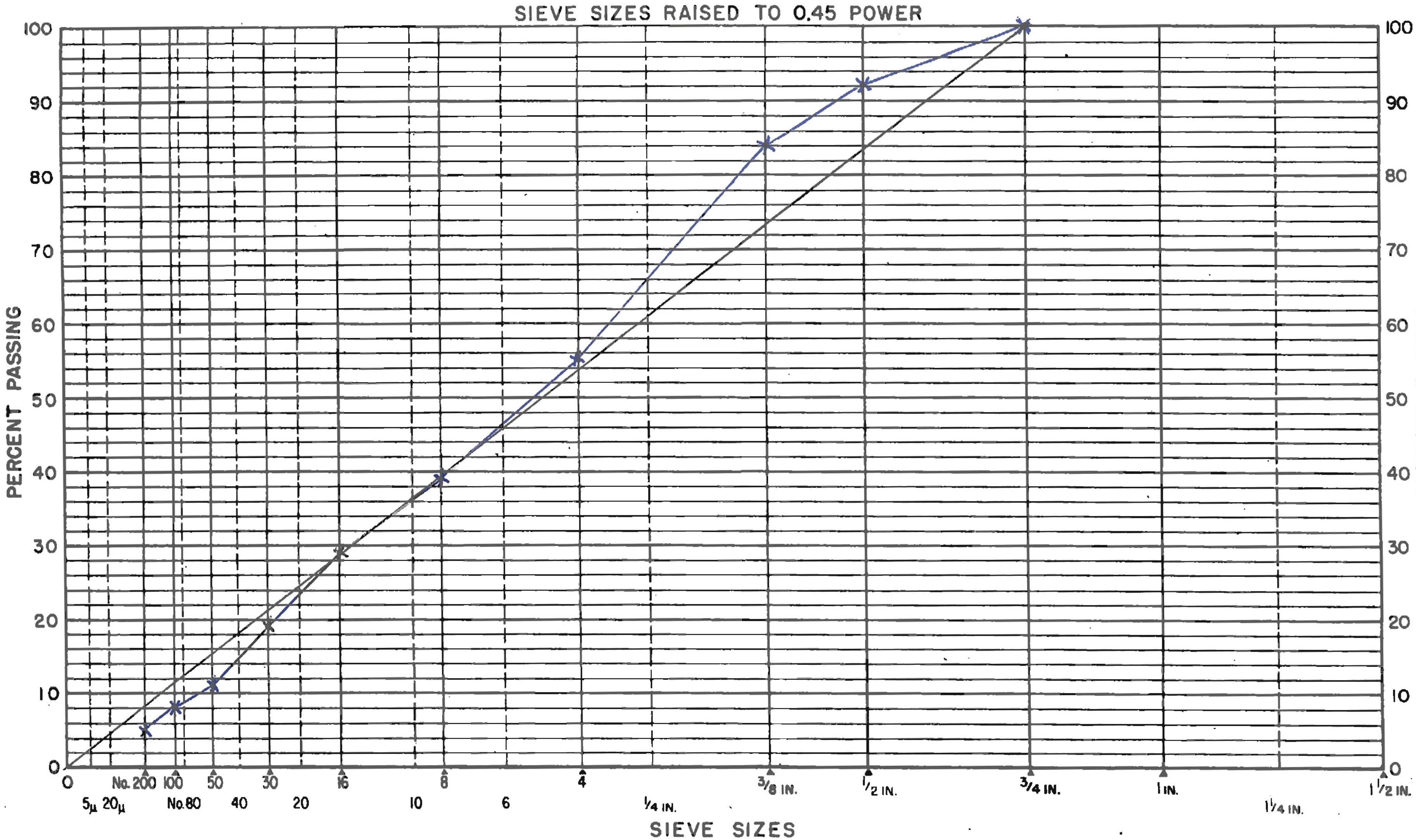
| Aging | Test | Test Temp. °C | Units of Results | Binder Type | | |
|---|---|---|---|---|---|---|
| | | | | AC-20R | AC-10P | |
| Tank | Sp.Gr. | 25 | | | | |
| | Flash | | °C | 288 | | |
| | Ab.Vis. | 60 | poises | | | |
| | Pen | 25 | dmm | | | |
| | DSR | 58 | kPa | 2.70 | 7.53 | |
| | | 64 | kPa | 1.34 | 3.77 | |
| | | 70 | kPa | 0.73 | 1.97 | |
| | | 76 | | | 1.05 | |
| | | 80 | | | 0.69 | |
| RTFOT | LOH | 163 | % | 0.06 | 0.20 | |
| | Ab.Vis. | 60 | poises | | | |
| | DSR | 58 | kPa | 2.83 | 6.85 | |
| | | 64 | kPa | 2.31 | 3.55 | |
| | | 70 | kPa | 1.15 | 1.82 | |
| PAV | DSR | 25 | kPa | 2,030 | 1,400 | |
| | | 22 | kPa | 2,980 | 2,010 | |
| | | 19 | kPa | 4,190 | 2,840 | |
| | | 16 | kPa | 5,700 | 4,000 | |
| | | 13 | kPa | | 5,400 | |
| | BBR Stiffness (S) | -12 | MPa | 155 | | |
| | | -18 | MPa | 327 | 192 | |
| | | -24 | MPa | | 369 | |
| | BBR Slope (m) | -12 | | 0.332 | | |
| | | -18 | | 0.265 | 0.309 | |
| | | -24 | | | 0.254 | |

# Appendix B:

# Gradations Used for the Laboratory Study

COLORADO DEPARTMENT OF HIGHWAYS

# GRADATION CHART

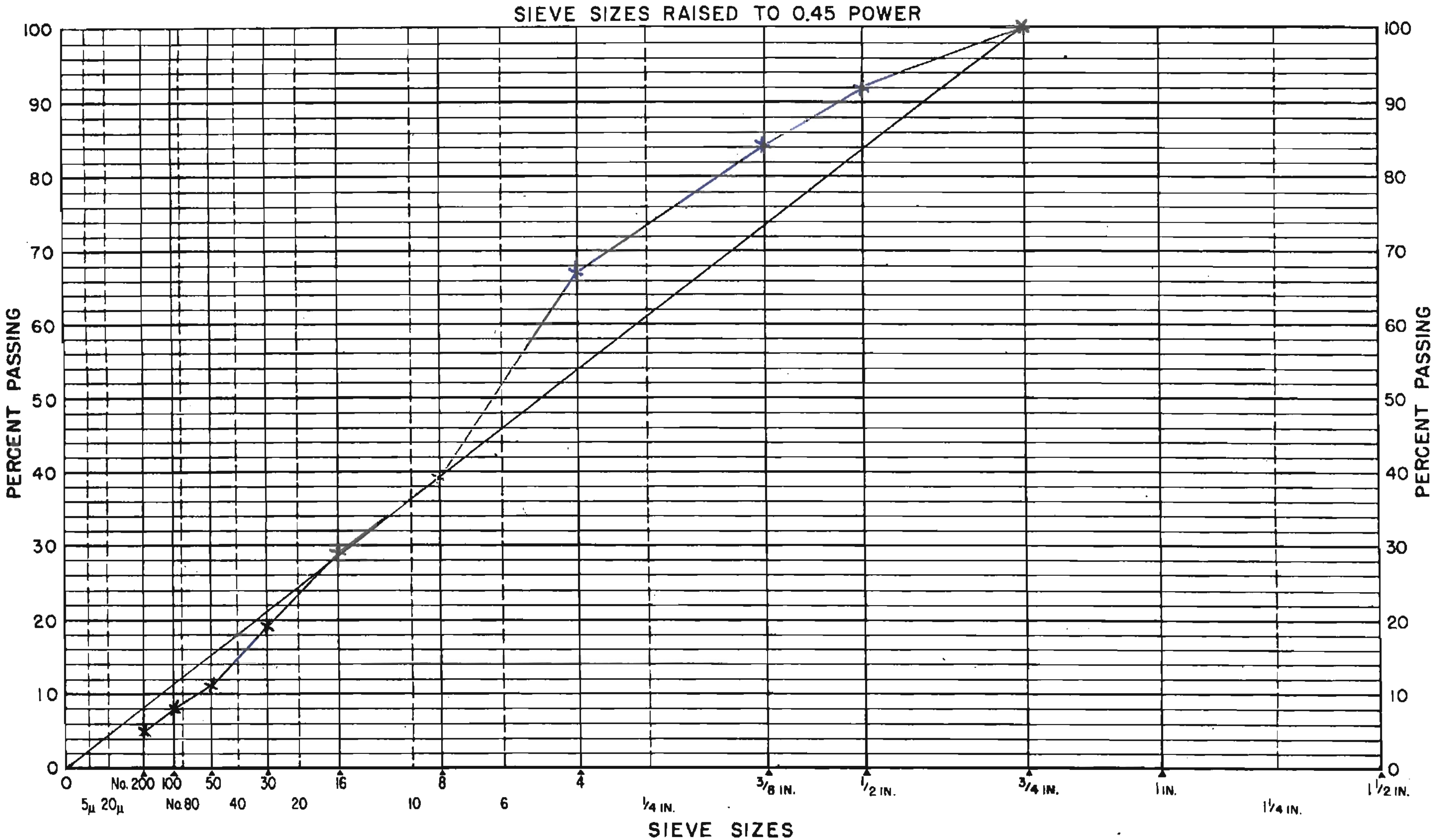



B-1

REMARKS: Mix 1 (Good Coarse - Good Fine)

COLORADO DEPARTMENT OF HIGHWAYS

# GRADATION CHART

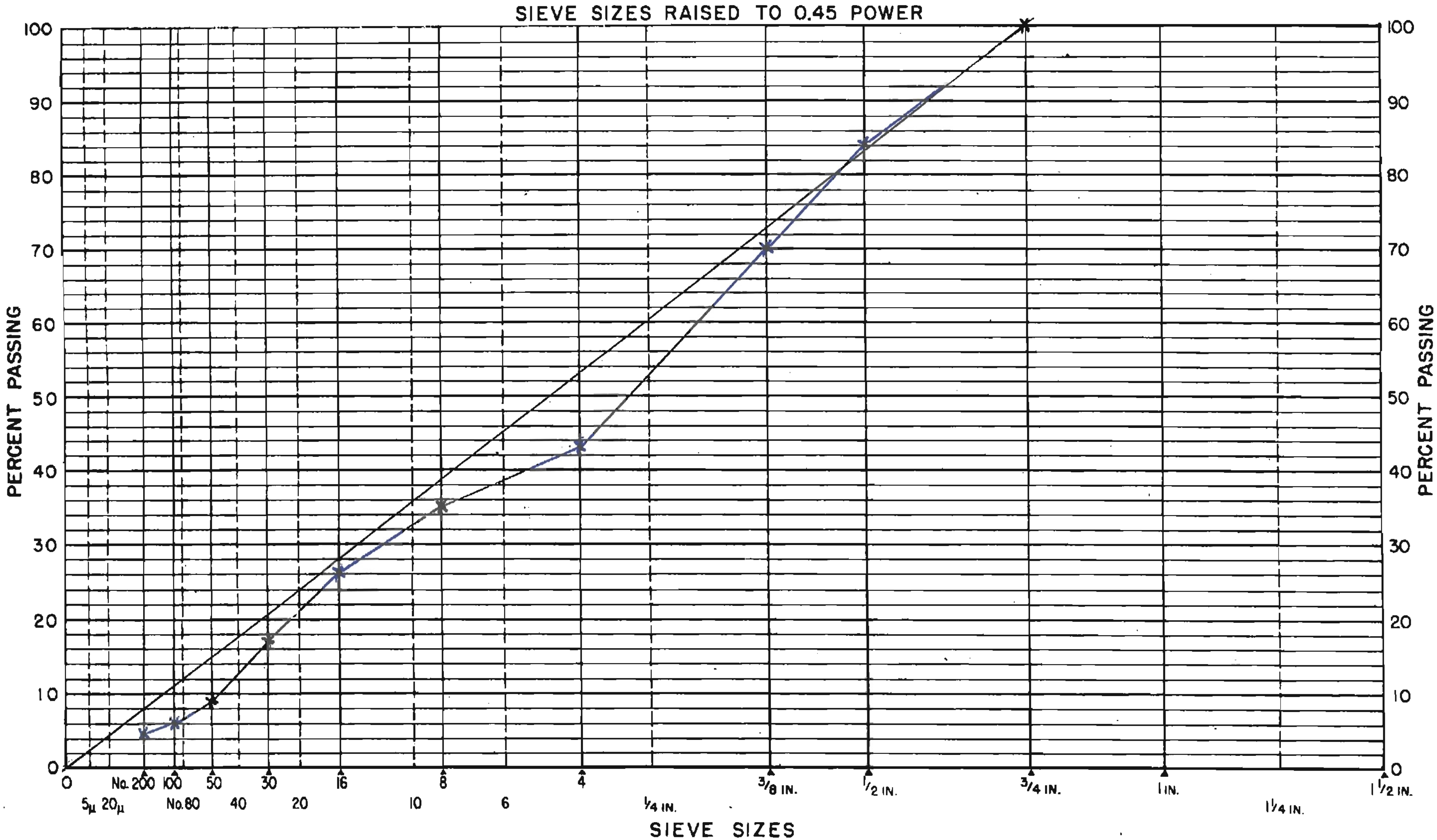



REMARKS:

Mix 2 ( Good Coarse - Poor Fine )

COLORADO DEPARTMENT OF HIGHWAYS

# GRADATION CHART



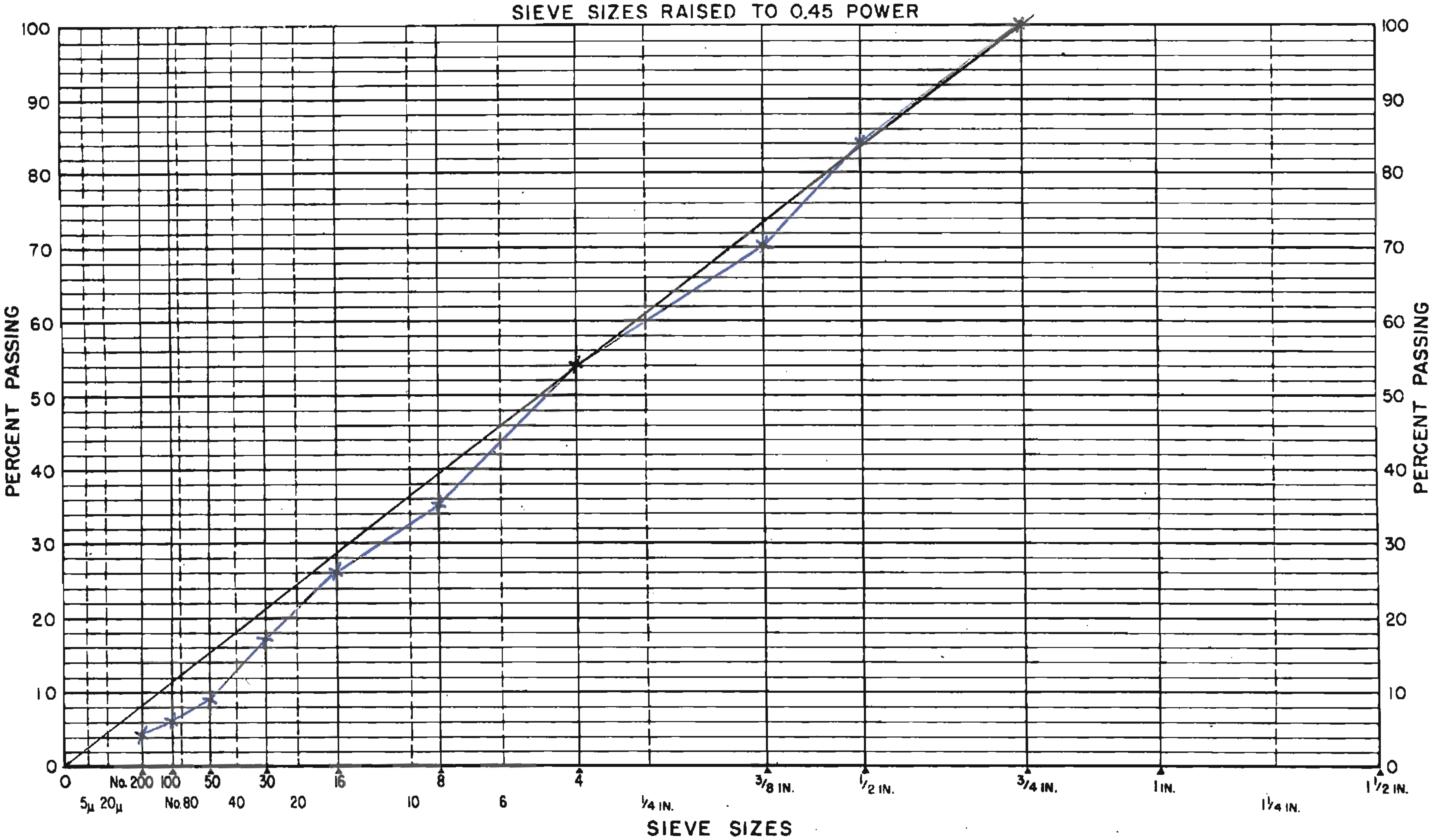

B-3

REMARKS:

Mix 3 ( Poor Coarse - Good Fine )

COLORADO DEPARTMENT OF HIGHWAYS

# GRADATION CHART




B-4

REMARKS:

Mix 4 (Poor Coarse – Poor Fine)

## Appendix C:

## SHRP Binder Test Results from the Field Experiment.

| Aging | Test | Test Temp. °C | Units of Results | Site 1 | Site 2 | Site 3 |
|---|---|---|---|---|---|---|
| Abson Recovery | DSR | 28 | kPa | | | 2720 |
| | | 25 | kPa | 4310 | 3280 | 3810 |
| | | 22 | kPa | 6330 | 4610 | 5030 |
| | | 19 | kPa | 8980 | 6630 | |
| | | 16 | kPa | | | |
| | | 13 | kPa | | | |
| | | 10 | kPa | | | |
| | BBR Stiffness (S) | -12 | MPa | 306 | 152 | 76 |
| | | -18 | MPa | 296 | 338 | 142 |
| | | -24 | MPa | | | 263 |
| | BBR Slope (m) | -12 | | 0.328 | 0.349 | 0.299 |
| | | -18 | | 0.253 | 0.273 | 0.276 |
| | | -24 | | | | 0.247 |

| Aging | Test | Test Temp. °C | Units of Results | Site | | |
|---|---|---|---|---|---|---|
| | | | | 4 | 6 | 7 |
| Abson Recovery | DSR | 28 | kPa | | | |
| | | 25 | kPa | 1140 | 3680 | 2180 |
| | | 22 | kPa | 1750 | 5040 | 3210 |
| | | 19 | kPa | 2550 | 6740 | 4710 |
| | | 16 | kPa | 3440 | | 6930 |
| | | 13 | kPa | 4700 | | |
| | | 10 | kPa | 6820 | | |
| | BBR Stiffness (S) | -12 | MPa | 71 | 147 | 92 |
| | | -18 | MPa | 128 | 376 | 199 |
| | | -24 | MPa | 293 | | |
| | BBR Slope (m) | -12 | | 0.498 | 0.448 | 0.379 |
| | | -18 | | 0.393 | 0.377 | 0.300 |
| | | -24 | | 0.301 | | |

| Aging | Test | Test Temp. °C | Units of Results | Site | | |
|---|---|---|---|---|---|---|
| | | | | 8 | 9 | 10 |
| Abson Recovery | DSR | 28 | kPa | | 2890 | 5090 |
| | | 25 | kPa | 2700 | 3920 | 7240 |
| | | 22 | kPa | 4080 | 5400 | |
| | | 19 | kPa | 6140 | | |
| | | 16 | kPa | | | |
| | | 13 | kPa | | | |
| | | 10 | kPa | | | |
| | BBR Stiffness (S) | -12 | MPa | 65 | 134 | 85 |
| | | -18 | MPa | 151 | 237 | 167 |
| | | -24 | MPa | | | |
| | BBR Slope (m) | -12 | | 0.379 | 0.281 | 0.301 |
| | | -18 | | 0.328 | 0.237 | 0.265 |
| | | -24 | | | | |