Arizona’s 15 Years of Experience Using the Four Point Bending Beam Test

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ABSTRACT: The Arizona Department of Transportation has used the Four Point Bending Beam Test since 1993 to evaluate the fatigue cracking and general cracking characteristics or tendencies of many different types of hot mix asphalt. The numerous mixes included dense graded, gap graded and open graded. In addition various asphalt binders were evaluated including several different performance graded asphalts and asphalt rubber. The mixes were evaluated at various air void levels and temperatures. All the evaluated mixes were done in conjunction with various highway paving projects constructed in a wide range of climate zones. The purpose of this paper is to report on the results of the testing and from this comment on the strengths and weakness of the testing method and possible improvements. Results of the testing may also be used to estimate cracking performance and to improve the predictive qualities of the analysis of test results.

1 PURPOSE OF PAPER

The purpose of this paper is to review the history of the experimental use of the four point bending beam (4PBB) test in the design of several paving projects constructed in Arizona from 1993 to the present. Also, to report on the results of the 4PBB testing associated with these projects designed and constructed from 1993 to the present. From this historical body of information and data, discuss the positive qualities and areas of concern of the 4PBB testing method as well as suggestions for possible improvements. Results of the testing may also be used to estimate cracking performance and to improve the predictive qualities of the analysis of test results.

2 HISTORY OF ARIZONA ASPHALT HOT MIX

The Arizona Department of Transportation (ADOT) has long been interested in improving the design, construction and performance of its pavements. Since the late 1960’s to the present the ADOT has used the Hveem, Marshall and Strategic Highway Research Program (SHRP) Superpave gyratory volumetric methods to design hot mix asphalt (HMA) pavements. The volumetric design criteria for typical 19mm HMA ranges include air voids of 5-7%, voids in mineral aggregate (VMA) of 15-18%. The Hveem stability generally was a minimum of 43 and the minimum Marshall stability was 9kN (2000 pounds) and the flow between 2-4mm (8 to 16; 0.01 inch). The asphalt binder has been graded by the penetration method as well as the AC and AR viscosity method. The ADOT has used the SHRP asphalt binder performance grading (PG) method since 1993 and all the projects reported on in this study reflect this asphalt binder grading method. Generally the field compaction is a minimum of 92% of the theoretical maximum density for typical HMA controlled by coring. For asphalt rubber mixes the compaction is controlled by means of nuclear gauge relative compaction in terms of a minimum number of roller passes. (Way, 1997).
3 FOUR POINT BENDING BEAM TEST

In 1993 SHRP recommended the repeated simple shear test at constant height (RSST-CH) or in this paper shear test to estimate future rutting and the four point bending beam (4PBB) test to estimate the fatigue cracking (Monismith, 1994). The ADOT became very interested in implementing both of these tests just as it supported the implementation of the asphalt binder PG grading system, aggregate consensus properties and the gyratory compactor (Sousa, 1995). However, both the shear test and 4PBB tests required very specialized and expensive equipment. Nevertheless, ADOT contracted with a private consulting testing service to conduct shear test and 4PBB testing on several experimental projects designed and built during 1993 to 1995. Later in about 1999 Arizona State University acquired the 4PBB equipment and another set of projects were designated for 4PBB tests in order to further advance the future implementation of the Mechanistic-Empirical Pavement Design Guide (MEPDG),(AASHTO, 2008).

3.1 4PBB Test background

This project used the 4PBB test equipment at the University of California at Berkeley (UCB) in a manner as described as part of the SHRP Research Project A-003A (Harvey, 1993). At that time in the early 1990’s 4PBB test was conducted with a very unique and expensive electromechanical-hydraulic loading device within an environment chamber. The device was designed and built by Cox in cooperation with input from UCB, Figure 1.

The cost of the equipment was about two million US dollars. The equipment required a large support apparatus for cooling, heating and pressure since it was designed to support both the 4PBB and the shear test. The sample of HMA for both the 4PBB and shear test was prepared by means of a novel rolling wheel compaction method that simulated the actual method of construction field compaction, Figure 2.
This was done by preparing and heating the properly weighed out HMA sample to the appropriate temperature and then placing it into a preheated metal mold of fixed dimensions. The sample was then rolled to a precise height such that the proper air void level achieved. After the mix within the mold cooled it was cored for the shear test sample and sawed to get the proper 4PBB sample. The shear test core and 4PBB sawed beam sample were further sawed and trimmed to obtain the proper size, geometry and surface. Care was taken to note the orientation of the sample (top and bottom) before testing began. The entire testing protocol was developed by UCB in line with all other SHRP research studies and actual pavement performance from various studies. Both the SHRP shear test and 4PBB test methods and analysis of test results was completed within the five year SHRP study period and offered to the highway community as a complete conceptual package, albeit not yet field tested or validated.

3.2 ADOT support of 4PBB testing 1993-1995

The ADOT wanted to field test and validate all aspects of the recommended SHRP HMA design method. As was done with the SHRP PG grading, consensus aggregate properties and gyratory compactor the ADOT also wanted to field test and validate the shear test and 4PBB. As previously noted the ADOT contracted with a consulting company to design and test HMA mixes in Arizona using the new shear test and 4PBB. The consulting company recognized the cumbersome and cost of the UCB testing equipment would make it impossible to be used in a field construction situation. Thus they invented a means of placing the shear test and 4PBB equipment, along with temperature control chamber, in a trailer and be used at a construction site. The ADOT contracted with the firm to do such testing on three major overlay projects from 1993 to 1995. One of the authors of this paper Dr. Sousa co-developed the equipment in the trailer with assistance from the Cox equipment company and was involved in the design and testing with this equipment in Arizona, Figure 3.
Figure 3. Trailer with Cox equipment to test RSST-CH, shear test and 4PBB specimens in the field

This testing included developing a mix design that balanced the rutting and fatigue cracking properties of the mix and to take field cores and beams to check the quality of the mix in relation to the design properties. Ultimately, the testing program was ended because of questions concerning the SHRP mix property testing for rutting and cracking. As a result of this suspension new research was nationally conducted to develop the MEPDG and its supporting tests which now include the 4PBB test for fatigue cracking.

3.3 ADOT support of Arizona State University 4PBB testing 1999 to the present

Following the earlier 1993-1995 4PBB efforts and subsequent national research the ADOT contracted with Arizona State University (ASU) to sample various project HMA mixes and conduct testing consistent with the new MEPDG pavement design procedure. This testing included 4PBB testing. The selected projects included SHRP gyratory mixes as well as Marshall design asphalt rubber gap graded (ARAC) and open graded (ARFC) mixes. The mixes are sampled as loose hot mix on the project and then brought back to the ASU laboratory for testing. Since the earlier testing conducted from 1993-1995 there has been much improvement in the testing protocol and documentation. The following discussion is an overview of the 4PBB testing method and protocol.

3.4 Background of the Flexural Beam Fatigue Test

Load associated fatigue cracking is one of the major distress types occurring in flexible pavement systems. The action of repeated loading caused by traffic induced tensile and shear stresses in the bound layers, which will eventually lead to a loss in the structural integrity of a stabilized layer material. Fatigue initiated cracks at points where critical tensile strains and stresses occur. Additionally, the critical strain is also a function of the stiffness of the mix. Since the stiffness of an asphalt mix in a pavement layered system varies with depth; these changes will eventually effect the location of the critical strain that varies with depth; these changes will eventually effect the location of the critical strain that causes fatigue damage. Once the damage initiates at the critical location, the action of traffic eventually causes these cracks to propagate through the entire bound layer.
Over the last 3 to 4 decades of pavement technology, it has been common to assume that fatigue cracking normally initiates at the bottom of the asphalt layer and propagates to the surface (bottom-up cracking). This is due to the bending action of the pavement layer that results in flexural stresses to develop at the bottom of the bound layer. However, numerous recent worldwide studies have also clearly demonstrated that fatigue cracking may also be initiated from the top and propagates down (top-down cracking). This type of fatigue is not as well defined from a mechanistic viewpoint as the more classical “bottom-up” fatigue. In general, it is hypothesized that critical tensile and/or shear stresses develop at the surface and cause extremely large contact pressures at the tire edges-pavement interface this, coupled with highly aged (stiff) thin surface layer that have become oxidized is felt to be responsible for the surface cracking that develops. In order to characterize fatigue in asphalt layers, numerous model forms can be found in the existing literature. The most common model form used to predict the number of load repetitions to fatigue cracking is a function of the tensile strain and mix stiffness (modulus). The basic structure for almost every fatigue model developed and presented in the literature for fatigue characterization is of the following form as shown in equation 1.

\[
N_f = K_1 \left( \frac{1}{\varepsilon_t} \right)^{K_2} \left( \frac{1}{E} \right)^{K_3} = K_1 \left( \varepsilon_t \right)^{-K_2} \left( E \right)^{-K_3} 
\]

where:

- \( N_f \) = number of repetitions to fatigue cracking
- \( \varepsilon_t \) = tensile strain at the critical location
- \( E \) = stiffness of the material
- \( K_1, K_2, K_3 \) = laboratory calibration parameters

In the laboratory, two types of controlled loading are generally applied for fatigue characterization: constant stress and constant strain. In constant stress testing, the applied stress during the fatigue testing remains constant. As the repetitive load causes damage in the test specimen the strain increases resulting in a lower stiffness with time. In case of constant strain test, the strain remains constant with the number of repetitions. Because of the damage due to repetitive loading, the stress must be reduced resulting in a reduced stiffness as a function of repetitions. The constant stress type of loading is considered applicable to thicker pavement layers usually more than 200 mm (8 inches). For AC thicknesses less than 200 mm (8 inches), fatigue behavior is governed by a mixed mode of loading, mathematically expressed as some model yielding intermediate fatigue prediction to the constant strain and stress conditions.

3.5 Testing Equipment

Flexural fatigue tests are performed according to the AASHTO T321-03 (AASHTO, 2003), and SHRP M-009 (SHRP, 1998). The flexural fatigue test has been used by various researchers to evaluate the fatigue performance of pavements (Harvey, 1993), (Tayebali, 1995). Figure 4 shows the flexural fatigue apparatus. The device is typically placed inside an environmental chamber to control the temperature during the test.

The cradle mechanism allows for free translation and rotation of the clamps and provides loading at the two center points as shown in Figure 5. Pneumatic actuators at the ends of the beam center it laterally and clamp it. Servomotor driven clamps secure the beam at four points with a pre-determined clamping force. Haversine or sinusoidal loading is applied to the beam via the built-in digital servo-controlled pneumatic actuator. The innovative “floating” on-specimen transducer measures and controls the true beam deflection irrespective of loading frame compliance. The test is run under either a controlled strain or a controlled stress loading. Note the AASHTO T321-03 states the following in relation to the application of the testing device loading and resultant deflection: “…The loading device shall be capable of (1) providing repeated sinusoidal loading at a frequency of 5 to 10 Hz, (2) subjecting specimen to 4-point
bending with free rotation and horizontal translation at all load and reaction points, (3) forcing the specimen back to its original position (i.e. zero deflection) at the end of each load pulse.”

Figure 4. ASU 4PBB testing apparatus

Figure 5. Loading characteristics of the 4PBB apparatus

In the constant stress mode, the stress remains constant but the strain increases with the number of load repetitions. In the constant strain test, the strain is kept constant and the stress decreases with the number of load repetitions. In either case, the initial deflection level is adjusted so that the specimen will undergo a minimum of 10,000 load cycles before its stiffness is reduced to 50 percent or less of the initial stiffness. In this study, all tests were conducted in the control strain type of loading.

3.6 Test Procedure and Calculations

The test utilized in this study applied repeated third-point loading cycles as was shown in Figure 5. The sinusoidal load was applied at a frequency of 10 Hz. The maximum tensile stress and maximum tensile strain were calculated as shown in equation 2 and 3:

\[ \sigma_t = \frac{0.357P}{b.h^2} \]  (2)
\[
\varepsilon_t = \frac{12.\delta h}{3.L^2 - 4.a^2}
\]

where,
- \(\sigma_t\) = Maximum tensile stress, Pa
- \(\varepsilon_t\) = Maximum tensile strain, m/m
- \(P\) = Applied load, N
- \(b\) = Average specimen width, m
- \(h\) = Average specimen height, m
- \(\delta\) = Maximum deflection at the center of the beam, m
- \(a\) = Space between inside clamps, 0.357/3 m; [0.119 m]; (4.7 in.)
- \(L\) = Length of beam between outside clamps, 0.357 m (14 in.)

The flexural stiffness was calculated using equation 4 as follows:

\[
S = \frac{\sigma_t}{\varepsilon_t}
\]

where,
- \(S\) = Flexural stiffness, Pa

The phase angle (\(\phi\)) in degrees was determined using equation 5 as follows:

\[
\phi = 360.f.s
\]

where,
- \(f\) = Load frequency, Hz
- \(s\) = Time lag between \(P_{\text{max}}\) and \(\delta_{\text{max}}\), seconds

The dissipated energy per cycle and the cumulative dissipated energy were computed using Equations 6 and 7, respectively.

\[
D = \pi.\sigma_t.\varepsilon_t.\sin(\phi)
\]

Cumulative Dissipated Energy = \(\sum_{i=1}^{N} D_i\)

where,
- \(w\) = Dissipated energy per cycle, J/m²
- \(w_i\) = \(w\) for the \(i^{th}\) load cycle

During the test the flexural stiffness of the beam specimen was reduced after each load cycle. The stiffness of the beam was plotted against the load cycles; the data was best fitted to an exponential function as follows:

\[
S = S_0 e^{bN}
\]
where,

\[ S = \text{Flexural stiffness after n load cycles, Pa} \]
\[ S_i = \text{Initial flexural stiffness, Pa} \]
\[ e = \text{Natural logarithm to the base e} \]
\[ b = \text{Constant} \]
\[ N = \text{Number of load cycles} \]

Once Equation 8 was formulated, the initial stiffness \( S_i \) can be obtained. Failure was defined as the point at which the specimen stiffness is reduced to 50 percent of the initial stiffness. The number of load cycles at which failure occurred, \( N_f \), was computed by solving Equation 9 as follows:

\[
N_{f,50} = \frac{\ln \left( \frac{S_{f,50}}{S_i} \right)}{b}
\]

\( N_{f,50} \) = Number of load cycles to failure
\( S_{f,50} \) = Stiffness at failure, Pa

3.7 Materials and Specimen Preparation

All beam specimens were prepared using the reheated hot mix asphalt mixes that were obtained during construction.

The AASHTO T321-2003 and SHRP M-009, flexural fatigue testing protocol, require preparation of oversize beams that later have to be sawed to the required dimensions. The final required dimensions are 380 ± 6 mm (15 ± 1/4 in.) in length, 50 ± 6 mm (2 ± 1/4 in.) in height, and 63 ± 6 mm (2.5 ± 1/4 in.) in width. The procedure does not specify a specific method for preparation. Several methods have been used to prepare beam molds in the laboratory including full scale rolling wheel compaction, miniature rolling wheel compaction, and vibratory loading.

In this study beams were prepared using vibratory loading applied by a servo-hydraulic loading machine. A beam mold was manufactured at ASU with structural steel that is not hardened. The mold consists of a cradle and two side plates as shown in Figure 6. The inside dimensions of the mold are 12 mm (1/2 inch) larger than the required dimensions of the beam after sawing in each direction to allow for a 6 mm (1/4 inch) sawing from each face. A top loading platen was originally connected to the loading shaft assembly in the middle as shown in Figure 7. Note that the top platen is made of a series of steel plates welded at the two ends to distribute the load more evenly during compaction. The loading shaft was connected to the upper steel plate rather than extending it to the bottom plate so that an arch effect is introduced that would assist in distributing the load more uniformly. In addition, it was found that if the bottom surface of the bottom plate is machined to be slightly concave upward, it would counter balance any bending that might occur during compaction and produce more uniform air void distribution.
The asphalt concrete mixture was heated for two hours at 163 °C (325 °F) and the conventional mixture was heated at 146 °C (295 °F). The mold was heated separately for one hour at the same temperature as the mix. The mixture was placed in the mold in one load. The mold was then placed on the bottom plate of the loading machine and the top platen was lowered to contact the mixture.

A small load of 1.4 kPa (0.2 psi) was then applied to seat the specimen. A stress-controlled sinusoidal load was then applied with a frequency of 2 Hz and a peak-to-peak stress of 2.8 MPa (400 psi) for the compaction process. Since the height of the specimen after compaction was fixed, the weight of the mixture required to reach a specified air void value was pre-calculated. Knowing the maximum theoretical specific gravity and the target air voids, the weight of the mixture was determined. During compaction the loading machine was programmed to stop when the required specimen height was reached. After compaction, specimens were left to cool to ambient temperature. The specimens were brought to the required dimensions for fatigue testing by sawing 6 mm (1/4 inch) from each side, Figure 8. The specimens were cut by using water cooled saw machine to the standard dimension of 63.5 mm (2.5 in.) wide, 50.8 mm (2.0 in.) high, and 381 mm (15 in.) long. Finally, the air void content was measured by using the saturated surface-dry procedure (AASHTO T166, Method A) for the conventional mixture.
3.8 Test factorial

One load mode (control strain) using 6 to 10 levels of strain, one replicate each was used for testing at 4.4, 21.1, and 37.8 °C (40, 70, and 100 °F). One of the most difficult tasks is to compact beams from field mixes so that they all have the same or tight range of air void levels. This may be possible, but would require a large amount of materials and many trials. Because of the variable strain levels selected and consequent regression analysis conducted, the air void variation was relaxed to accept samples that are within 1% range.

3.9 Test Conditions

In summary the following conditions were used:
- Air voids: 7% for the conventional 1 & 2 mixtures.
- Load condition: Constant strain level, 6 – 10 levels of the range (300-1300 μ strain)
- Load frequency: 10 Hz
- Test temperature: 4.4, 21.1, and 37.8 °C (40, 70, and 100 °F) for conventional dense graded mixtures.

The tests were performed according to the AASHTO T321-2003, and SHRP M-009 procedures. Initial flexural stiffness was measured at the 50th load cycle. Fatigue life or failure under control strain was defined as the number of cycles corresponding to a 50% reduction in the initial stiffness. The loading on most specimens was extended to reach a final stiffness of 30% of the initial stiffness instead of the 50% required by AASHTO T321-2003 and SHRP M-009. The control and acquisition software load and deformation data were reported at predefined cycles spaced at logarithmic intervals.

4 RESULTS OF FOUR POINT BENDING BEAM TESTING

4.1 Projects and materials

Since 1993 several projects have been sampled and tested. Figure 9 shows their approximate location. The projects along with their traffic loading in 18 kip single axle loads per year are shown in Table 1. The projects are mostly overlay projects of varying thickness from 50mm to 100 mm in thickness. All the projects have a nominal 19 mm thick ARFC surfacing except for the Perryville project which did not have an ARFC surface. The projects represent various asphalt mixes including dense graded HMA, gap graded-coarse graded asphalt rubber (ARAC) and open graded asphalt rubber (ARFC). An average gradation of each type of mix is shown in Table 2, along with their respective average binder content, average air voids and average VMA.
Table 1. Project location and traffic loading

<table>
<thead>
<tr>
<th>Project ID</th>
<th>Name</th>
<th>Route</th>
<th>Milepost Begin &amp; End</th>
<th>Year Built</th>
<th>ESAL’s/yr (000’s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Deer Valley</td>
<td>I-17</td>
<td>224.5-225.8 NB</td>
<td>1993</td>
<td>1000</td>
</tr>
<tr>
<td>2</td>
<td>Perryville</td>
<td>I-10</td>
<td>112.3-122.3 WB</td>
<td>1995</td>
<td>3000</td>
</tr>
<tr>
<td>3</td>
<td>Sedona</td>
<td>I-17</td>
<td>299.0-311.6 NB</td>
<td>1995</td>
<td>600</td>
</tr>
<tr>
<td>A</td>
<td>Antelope Wash</td>
<td>US 93</td>
<td>95.1-101.9</td>
<td>2006</td>
<td>200</td>
</tr>
<tr>
<td>B</td>
<td>Burro Creek</td>
<td>US 93</td>
<td>138.0-142.5</td>
<td>2006</td>
<td>200</td>
</tr>
<tr>
<td>C</td>
<td>Buffalo Range</td>
<td>I-40</td>
<td>224.7-229.9</td>
<td>2001</td>
<td>2200</td>
</tr>
<tr>
<td>D</td>
<td>Salt River</td>
<td>Int.</td>
<td>Various</td>
<td>2001</td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>Silver Springs</td>
<td>I-40</td>
<td>79.5-86.2</td>
<td>2006</td>
<td>1500</td>
</tr>
<tr>
<td>F</td>
<td>Badger Springs</td>
<td>I-17</td>
<td>256.0-263.0</td>
<td>2005</td>
<td>600</td>
</tr>
<tr>
<td>G</td>
<td>Kohl’s Ranch</td>
<td>SR 260</td>
<td>266.0-269.0</td>
<td>2007</td>
<td>100</td>
</tr>
<tr>
<td>H</td>
<td>Two Guns</td>
<td>I-40</td>
<td>230.0-240.0</td>
<td>2003</td>
<td>1500</td>
</tr>
<tr>
<td>I</td>
<td>Jackrabbit</td>
<td>I-40</td>
<td>268.0-277.4</td>
<td>2003</td>
<td>2000</td>
</tr>
</tbody>
</table>
Table 2. Average percent passing and various mix properties

<table>
<thead>
<tr>
<th>Sieve mm</th>
<th>Dense HMA</th>
<th>ARAC</th>
<th>ARFC</th>
</tr>
</thead>
<tbody>
<tr>
<td>25.40</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>19.00</td>
<td>98</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>12.70</td>
<td>81</td>
<td>82</td>
<td>100</td>
</tr>
<tr>
<td>9.50</td>
<td>69</td>
<td>73</td>
<td>100</td>
</tr>
<tr>
<td>6.40</td>
<td>57</td>
<td>49</td>
<td>68</td>
</tr>
<tr>
<td>4.75</td>
<td>50</td>
<td>37</td>
<td>36</td>
</tr>
<tr>
<td>2.36</td>
<td>36</td>
<td>20</td>
<td>7</td>
</tr>
<tr>
<td>2.00</td>
<td>32</td>
<td>18</td>
<td>6</td>
</tr>
<tr>
<td>1.18</td>
<td>25</td>
<td>12</td>
<td>4</td>
</tr>
<tr>
<td>0.60</td>
<td>17</td>
<td>8</td>
<td>3</td>
</tr>
<tr>
<td>0.42</td>
<td>14</td>
<td>6</td>
<td>2</td>
</tr>
<tr>
<td>0.30</td>
<td>11</td>
<td>5</td>
<td>2</td>
</tr>
<tr>
<td>0.15</td>
<td>6</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>0.075</td>
<td>4</td>
<td>1.7</td>
<td>1.2</td>
</tr>
<tr>
<td>% Binder</td>
<td>4.9</td>
<td>7.0</td>
<td>9.2</td>
</tr>
<tr>
<td>% Air Voids</td>
<td>6.6</td>
<td>8.6</td>
<td>18</td>
</tr>
<tr>
<td>% VMA</td>
<td>16.0</td>
<td>20.0</td>
<td>32.0</td>
</tr>
</tbody>
</table>

4.2 4PBB test results

For each project a full report was prepared with considerable detailed information (Sousa, APT, 1993-1995), (Kaloush, ASU, 2002-2008). It is beyond the scope of this report to go into the full detail of all the reported tests, however, virtually all the pertinent summary 4PBB information is described in this section. The 4PBB tests were conducted at the appropriate estimated fatigue temperature or range of temperatures. The k1 and k2 coefficients are determined from laboratory tests on the beams and are part of the following fatigue equation 10.

\[ N_f = k_1 \times (\varepsilon_t)^{k_2} \]  

\( \varepsilon_t \) = tensile strain at the critical location. Strain is in actual test units, example 0.000100 inch/inch.

\( N_f \) is the number of load cycles which represent a surrogate for 18 kip Equivalent Single Axle Loads which ultimately lead to fatigue cracking in the pavement.

Where the k1 and k2 coefficients are the values found from the testing and represent the relationship between the ESAL’s to failure value \( N_f \) related to the corresponding measured actual strain. This relationship gives a straight line on a log/log plot of load cycles, \( N_f \), to failure versus the strain as shown in the illustrative example, Figure 10.
The form of Equation 10 was used in this study rather than Equation 1 so that the 1993 4PBB test results could be compared to the results obtained later by ASU. In 1993 the stiffness term was not incorporated into the Fatigue equation. Table 3 shows the 1993-1995 4PBB test results in accordance with the SHRP project A-003(4), SHRP report A-404 (SHRP, 1994). This test method is similar to AASHTO provisional standard TP 321-03 (AASHTO, 2003). During design at least five test specimens were prepared and a straight line fitted to the test results. The correlation $R^2$ values were typically greater than 0.90. Table 4 shows the ASU 4PBB test results. For these test results at least six beams are tested at various strain levels at constant temperature and a straight line fitted to the data. The correlation $R^2$ values typically are above 0.90, see Figure 10.

Figure 10. Example number of load cycles versus strain at 50 percent of initial stiffness. Each data point is a separately compacted beam.

Table 3. 4PBB test results for 1993-1995 projects

<table>
<thead>
<tr>
<th>Site</th>
<th>Mix Type</th>
<th>PG/AC</th>
<th>%</th>
<th>%</th>
<th>Binder</th>
<th>Bind</th>
<th>Air</th>
<th>Test</th>
<th>k1</th>
<th>k2</th>
<th>Calc. Nf at $\varepsilon = 0.0001$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SHRP 70-10</td>
<td>4.0</td>
<td>4.0</td>
<td>20</td>
<td>5.56E-17</td>
<td>-6.13</td>
<td>1.84E+08</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>SHRP 70-10</td>
<td>5.0</td>
<td>4.0</td>
<td>20</td>
<td>6.35E-16</td>
<td>-6.04</td>
<td>9.18E+08</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>1</td>
<td>SHRP H 70-10</td>
<td>4.0</td>
<td>7.4</td>
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<td>3.48E-11</td>
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Table 4. 4PBB ASU test results

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<th>Bind</th>
<th>% Air</th>
<th>Voids</th>
<th>Temp</th>
<th>k1</th>
<th>k2</th>
<th>Calc. Nf at e = 0.0001</th>
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Table 5 is a summary of the ADOT percent cracking for each of the projects under review. The percent cracking shown in Table 5 is representative of the degree of cracking shown in Figure 11. The percent cracking method used by the ADOT was developed in the 1970’s and has been used consistently by the ADOT as part of their pavement management pavement performance method since 1980. (Way, 1979). Percent cracking values are recorded annually along with the rut depth, skid resistance, ride smoothness and maintenance at each routes milepost marker which represents approximately 7400 measurements per year since 1980. Although not shown in this report the rut depth and skid resistance for all the projects is at a satisfactory level of service. As can be seen in Table 5 the Perryville project, number 2, pavements with low binder content and no ARFC surface have the greatest degree of cracking. The ARAC section on the Perryville project, number 2, with a high percent asphalt binder has no cracking in addition it has virtually no rutting and good skid resistance. All of the other projects have an ARFC surfacing and their percent cracking is 3 percent or lower.

Figure 11. Example load cycles versus strain
Table 5. Cracking performance of experimental projects

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5 4PBB EQUIPMENT CORRELATION STUDY

The 4PBB test results reported in 1993-1995 and then later by ASU were measured with different equipment. The 1993 testing were done with a Cox 4PBB built by James Cox and Sons, Inc., in Colfax, California (Cox, 2009). Presently this same equipment is sold by Cox as Model CS7600 Asphalt Fatigue Testing Fixture. The ASU equipment was acquired in the late 1990’s from Industrial Process Controls Ltd. This company changed their name in June 2002 to IPC Global which is located in Victoria, Australia, (IPC, 2009). The IPC 4PBB equipment is sold as the Beam Fatigue Apparatus. As a part of this long term study Consulpav and ASU agreed that some degree of comparison between both devices was needed. ASU took some of the gap graded asphalt rubber (ARAC) mix from one of the ADOT test projects and prepared duplicate beams at approximately the same air void content. Half of the beams were tested at the ASU, Tempe, Arizona laboratory and the other half shipped to Lisbon, Portugal for Consulpav to conduct their testing. The selected ARAC mix had a PG 58-22 binder with a binder content of approximately 7.5 percent by weight of the mix. ASU tested its beams for air voids with the Corelok (Corelok, 2009) device and Consulpav with para-film after the method developed by UCB. ASU found the beams to have a 10.3 percent level of air voids and Consulpav found their beams to have an air void of 9.7 percent. Comparative results of pertinent 4PBB measurements are shown in Figure 12. Although the equipment is made by different manufacturers and samples were transported over the ocean the test results appear reasonably close.
6 DISCUSSION OF FOUR POINT BENDING BEAM TEST RESULTS

The use of the 4PBB test as a pavement analysis tool and/or design tool has been under research investigation since at least the late 1980’s. Over the years the manner in which this tool is used to estimate or predict fatigue cracking should be used has evolved. Virtually all of the original SHRP testing at University of California at Berkeley (UCB) involved making the beams with a rolling wheel compactor in a manner to simulate the actual construction method of compaction. This same technique was used for the first three ADOT projects in the early 1990’s. However, the rolling wheel compaction method of compaction did not fit well with the recommended gyratory method of compaction. In addition the cost of the equipment to do both the 4PBB and shear test appeared prohibitive, albeit the trailer mounted equipment was less expensive than the original UCB device and portable.

Another limitation was the time it took to test a beam sample and obtain the results for construction quality control seemed too long. Later via the trailer it was found that the beam taken from the field could be tested at a relatively high rate of strain thus reducing the number of bending cycles. The resultant test values could be converted to the number of cycles at a lower strain level more representative of the actual strain in the field. Nevertheless, by the mid 1990’s the 4PBB test was relegated to further research and efforts to use it for design and construction quality control.

In the late 1990’s the ADOT worked with ASU to further the research and the development of the 4PBB test to characterize the cracking potential of various Arizona mixes. ASU developed a vibratory means of preparing the beams from samples taken from the field. From construction test results the beams were compacted to the field voids and tested at three temperatures. The testing protocol and performance modeling advanced via the MEPDG. However, some testing issues still need to be further refined. Actual beams from the field are difficult to obtain because pavements are typically placed just a little too thin in thickness to make a suitable beam for testing. Also, some mixes such as the gap graded and open graded asphalt rubber mixes are placed 50mm to 12.5mm (2 inch to 0.5 inch) in thickness which is too thin to sample.
as sawed beam in the field and test. The beam in laboratory has some limited confinement but for open graded mixes it is difficult to perform a high temperature test without confinement.

Even with these issues the 4PBB test does offer many positive benefits in terms of estimating fatigue cracking. In reviewing the data in Tables 3 and 4 the asphalt rubber mixes with higher binder content consistently give a longer fatigue cracking life. This prediction estimate at a cursory level matches up well with the actual cracking observed in Table 5. All the projects with asphalt rubber higher binder contents have yet to reach the 10 percent level of cracking. In fact it is rare for an asphalt rubber mix with high binder content to reach the 10 percent cracking level as shown in Figure 13. Those test sections with the least percent asphalt binder (3.8 and 4.3% binder) on the Perryville project, (number 2) and no asphalt rubber wearing course reached and exceeded the 10 percent level. 4PBB test results for these mixes indicated a shorter fatigue life than the asphalt rubber higher binder mixes which indeed was the case.

Although these are only a small example of what the 4PBB test can predict they are a start and will greatly help to reinforce confidence in using this test method in the future. Much work still remains in how to interpret and integrate the 4PBB test into a practical pavement design and mix control procedure.

Figure 13. General cracking trend of ADOT dense graded with no asphalt rubber and asphalt rubber mixes

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