Impact of High RAP Content on the Performance Characteristics of Asphalt Mixtures in Manitoba

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ABSTRACT

Pavement sections with 15% RAP, 50% RAP with and without virgin binder grade change, and a conventional hot-mix without RAP were built side-by-side in 2009 on Provincial Trunk Highway 8 from Gimli to Hnausa in Manitoba, Canada. During construction, field-produced mixtures and raw materials were sampled for further evaluation. The raw materials were used to reproduce the various mixtures in the laboratory. This paper presents the results of an extensive laboratory evaluation of the field- and laboratory-produced mixtures to moisture damage and thermal cracking resistance. The moisture damage was evaluated using the dynamic modulus test at multiple freeze-thaw cycles. The thermal cracking resistance of the mixtures was also evaluated at multiple freeze-thaw cycles using the thermal stress restrained specimen test (TSRST). Overall, HMA mixtures with 50% RAP resulted in acceptable resistance to moisture damage and thermal cracking. The use of multiple freeze-thaw cycles provided better indication of the mixture resistance to moisture damage. Overall, the properties of the laboratory-produced mixtures in terms of moisture damage and thermal cracking resistance can be used to ensure quality field-produced mixtures.

INTRODUCTION

The use of recycled asphalt pavements (RAP) in asphalt mixtures can significantly reduce greenhouse gas emissions by eliminating the considerable fuel consumption required to acquire and process raw materials for virgin mixtures [1]. Hence, the increase use of RAP in asphalt mixtures makes asphalt pavements more sustainable providing the in-service pavement condition and performance are not jeopardized. Although, many highway agencies have realized the benefits of using RAP, the incorporation of high percentage of RAP (more than 25%) in hot mix asphalt (HMA) mixtures, especially in the asphalt surface layer, has been relatively low compared to the possible supply of RAP. This has been mainly attributed to concerns with durability and crack resistance of HMA mixtures with high RAP content.

At low RAP contents, there may not be a sufficient amount of old material to significantly affect the properties of the virgin asphalt binder but, at higher RAP contents, the hardened RAP binder may stiffen the mix and more likely improve the mixture resistance to rutting but not its resistance to cracking. Consequently, well developed design methods and analysis of the material properties for HMA mixtures with high RAP contents are deemed necessary.

The objective of the mix design process of HMA is to develop an economical blend of aggregates and asphalt that meet the design requirements and provide a good understanding of how a particular mix will perform in the field during construction and under subsequent traffic and environmental loadings. Therefore, it is critical to assess the impact of the various mix components, including RAP, on the performance of the constructed pavement (i.e. resistance to rutting, fatigue, and thermal cracking). The key to successfully include RAP in the HMA mix is the ability to assess its impact on pavement’s performance while recognizing the uniqueness of each project with respect to both materials and loading conditions.

Moisture damage of asphalt mixtures is very prominent throughout the U.S. and Canada and is not limited to the cold and wet regions. Moisture damage can be perceived as a process that destroys the integrity of the mix and makes it susceptible to multiple distress modes (e.g. rutting, fatigue, thermal cracking, and raveling) that are accelerated by the combined impact of environment and traffic loads. Hence, moisture damage is very critical to the long-term field performance of asphalt pavement and highway agencies have been confronting this issue by using various types of anti-strip additives.

During plant production, RAP material is introduced at ambient temperature to superheated virgin aggregate. Hence, concerns were raised for the effect of RAP on the moisture susceptibility of the mix; especially that RAP stockpile tends to hold water and has inferior drainage capabilities than normal aggregate stockpile. Whenever exists, the high moisture content in the RAP stockpile is expected to be removed during production when mixing the ambient RAP with superheated aggregates. This may require an increase in fuel usage and a reduction in production rate when high RAP is used.

In 2010, Hassan [2] showed that 40% RAP is the maximum feasible content with the available recycled hot asphalt technologies. Higher contents of RAP would require the use of indirect heat techniques or warm asphalt technology and involve more processing and testing of RAP to reduce variability. The use of rejuvenating agents also allows for higher contents of RAP to be incorporated in HMA mixes. However, these options are associated with additional costs that not all authorities or suppliers would be willing to incur.

Significant efforts have been performed by Federal Highway Administration, the National Center for Asphalt Technology (NCAT) and various Expert Task Groups to promote the use of mixtures with high RAP content by working with State Department of Transportation on field projects to showcase the performance of high RAP mixtures [3].

In 2009, NCAT [4] compared virgin and recycled asphalt pavements (6 to 17 years old) using data from 18 Long Term Pavement Performance (LTPP) projects across North America. Distress parameters from paired sections of virgin asphalt mix and recycled asphalt mix containing 30% RAP were analyzed. It was found that the performance of recycled and virgin sections were not statistically significantly different except for fatigue, longitudinal cracking, and transverse cracking, where the virgin sections performed slightly better overall than the RAP sections. NCAT concluded that, in most cases, using 30% RAP in an asphalt pavement can provide the same overall performance as virgin asphalt pavement [4].

In an attempt to encourage the construction of more sustainable asphalt pavements, the Manitoba Infrastructure and Transportation (MIT) in collaboration with the Asphalt Research Consortium (ARC) built, in 2009, pavement sections with high RAP contents on Provincial Trunk Highway 8 in Manitoba, Canada. The pavement sections are intended to provide necessary information for assessing the feasibility of using HMA Mixtures with high RAP content in surface layers in cold weather regions such as Manitoba, Canada.

OBJECTIVE

The objective of this paper is to evaluate and assess the impact of RAP content on the mixture resistance to moisture damage and thermal cracking. Additionally, the performance of the field-produced mixtures was compared to laboratory-produced mixtures.

PROJECT DESCRIPTION

The project is located on provincial highway 8 between Gimli and Hnausa in Manitoba, Canada. The total project length is about 17 miles, with the comparative pavement site accounting for 6.0 miles of the project. The traffic information for the project was provided by MIT and is shown in Table 1. The 20 year design traffic consisted of 1,950,000 equivalent single axle load (ESAL).

The evaluated sections were constructed in September 2009 and consisted of two 2-inch lifts with conventional HMA (i.e. 0% RAP), 15% RAP, and 50% RAP with no grade change for the new asphalt and 50% RAP with a grade change for the new asphalt. The pavement sections were laid on top of a 4-inch HMA with 50% RAP that was constructed the year before (i.e. 2008), which is on top of a base and a subgrade. All four evaluated mixes in the top two lifts consisted of a dense graded asphalt mixture manufactured with a Pen 150-200 asphalt binder except for the 50% RAP mix with grade change that was manufactured with a Pen 200-300 asphalt binder. The target binder grade for the project location was Pen 150-200.

During construction of the various sections the same hot plant (central feed plant), truck type, paver, crews, and compaction equipment were employed. During production, significant effort was made by the MIT staff to ensure the correct asphalt binder content and type was used for each test section and to ensure the lifts were constructed on top of each other. The compaction effort applied to both the conventional HMA (i.e. 0% RAP) and RAP sections was identical. Target in-place densities were achieved in the various sections without any noticeable issues.

EXPERIMENTAL DESIGN

Loose mixtures were sampled during paving of the top lift from the paving auger at the project site. Those mixtures were referred to as field-produced mixtures and are labeled F-0%-150, F-15%-150, F-50%-150 and F-50%-200. For instance, The F-0%-150 mix represents the field mix with 0% RAP manufactured with the Pen 150-200 asphalt binder while the F-50%-200 mix represents the field mix with 50% RAP manufactured with the Pen 200-300 asphalt binder. Additionally, cold feed aggregates, asphalt binders and RAP materials for the various mixtures were sampled during production at the plant location. The raw materials were used to reproduce all four mixtures in the laboratory. Those mixtures were referred to as laboratory mixtures and are labeled L-0%-150, L-15%-150, L-50%-150 and L-50%-200.

Table 2 shows the experimental program for this study. The program builds on the basis of testing field-produced and laboratory-produced HMA mixtures as they are subjected to moisture conditioning. The moisture conditioning consisted of subjecting the samples to multiple freeze-thaw (F-T) cycling. The moisture sensitivity of the mixtures was evaluated using the unconditioned and moisture-conditioned indirect tensile strengths (TS) along with the indirect tensile strength ratio (TSR) at multiple F-T cycles. Additionally, all mixtures were evaluated in terms of their mechanical property using the dynamic modulus test and their resistance to thermal cracking using the thermal stress restrained specimen test (TSRST) after multiple F-T cycles.

ASPHALT MIX DESIGNS

The Marshall Mix Design method as outlined in the Asphalt Institute's Mix Design Methods Manual MS-2 was used to design the field-produced and laboratory-produced mixtures following MIT standard specifications. All evaluated mixtures were designed with 75 blows on each side. A sieve analysis performed on extracted aggregates (AASHTO T308 and T30 [5]) from all four field-produced mixtures revealed that all aggregate gradations met the job mix formula criteria (see Table 3). The asphalt binder contents of RAP material and field-produced mixtures were determined using the ignition oven method (AASHTO T308 [5]). An asphalt binder content of 4.7% was measured for the RAP material. A binder content of 5.1% was measured for the 0 and 15% field-produced mixtures (i.e. F-0%-150 and F-15%-150) while a binder content of 4.9% was measured for the 50% field-produced mixtures (i.e. F-50%-150 and F-50%-200). All extracted asphalt binder contents were within 0.1% of the design binder contents.

ASPHALT BINDERS GRADES

The Superpave Performance Grading (PG) binder system (AASHTO M320 [5]) was used to grade the virgin binders, RAP binder and recovered blended binders from the various field-produced and laboratory-produced loose mixtures. All recovered binders were extracted using a centrifuge (AASHTO T164 [5]) and recovered using a rotary evaporator (ASTM D5404 [6]) and a solution that consists of 85% Toluene and 15% Ethanol by volume. The recovered binders were graded by testing them as original, short-term aged through the Rolling Thin Film Oven (RTFO), and long-term aged through the Pressure Aging Vessel (PAV). It should be noted that all laboratory-produced loose mixtures were subjected to short-term aging for four hours at 275°F in a forced draft oven while field-produced mixtures were not. Figure 1 summarizes the critical temperatures of the various binders. Critical temperatures are temperatures at which a binder just meets the appropriate specified Superpave criteria. Table 4 summarizes the Superpave PG grades. Looking at the data the following findings can be observed:

- The low critical temperatures of both virgin binders (i.e. Pen 150-200 and Pen 200-300) were only within 2°C of each others. However, the high critical temperatures were within 5°C of each others.
- In the case of both, field-produced and laboratory-produced mixtures manufactured with PG58-28 asphalt binder, the increase in RAP content in a mix resulted in a higher and warmer high and low critical temperatures for the recovered asphalt binders, respectively.
- On average, regardless of the RAP content, the recovered blended asphalt binders from field-produced mixtures had warmer critical temperatures than blended asphalt binders recovered from the laboratory-produced mixtures. On average, the high and low critical

temperatures of the blended asphalt binders recovered from the field-produced mixtures were higher by about 2.4 and 1.2°C, respectively. In other words, the asphalt binders recovered from field-produced mixtures were stiffer than those recovered from laboratory-produced mixtures. This indicates that the four hours of aging in a forced draft oven at 275°F did not simulate the aging of the field-produced mixtures.

- The recovered blended asphalt binders from the F-50%-200 and L-50%-200 mixtures were softer than the recovered blended asphalt binders from the F-50%-150 and L-50%-150 mixtures by about 4.0 and 4.2°C, respectively.

Overall, the recovered binders from the mixtures containing 0 and 15% RAP met the target grade for the project location of PG58-28. The recovered binders from the mixtures with 50% RAP met or exceeded the target high PG of 58°C but failed to meet the target low PG of -28°C. This observation was true for both mixtures with and without grade change. The use of softer asphalt binder (i.e. PG52-34) with the 50% RAP mix did not improve the low performance temperature of the blended asphalt binder in the mix enough to the point of meeting the target low PG. As mentioned before, the low critical temperature of both virgin binders were only within 2°C of each others. Due to the short supply of different types of asphalt binders in the vicinity of the project, the only available commercial soft binder was used in this study (i.e. Pen 200-300). It should be noted that MIT has been using the Pen 200-300 asphalt binder in their asphalt base layers with mixtures containing up to 70% RAP.

MIXTURES EVALUATION

AASHTO T283 Test at Multiple F-T Cycles

The moisture sensitivity of the various mixtures was evaluated using the unconditioned and moisture-conditioned indirect tensile strengths (TS) along with the indirect tensile strength ratio (TSR) at multiple F-T cycles. The multiple F-T cycling followed the procedure outlined in AASHTO T283 [5] at multiple stages. For each mixture, a total of fifteen 4-inch diameter samples were compacted using the Marshall compactor to 7±0.5% air voids. The samples were divided into three subsets of five samples each: unconditioned subset (i.e. 0 F-T), moisture-conditioned subset to 1 F-T, and moisture-conditioned subset to 3 F-T. The 3 F-T cycles represented the moisture damaged stage as some of the mixtures started to disintegrate after four F-T cycles. Each mixture was evaluated following the procedure outlined below:

- Measure the TS of the unconditioned subset (i.e., 0 F-T cycles).
- Subject the five samples of each of the second and third subsets to 75±5% saturation.
- Subject the saturated samples to the required number of freeze-thaw cycling wherein one freeze-thaw cycle consists of freezing at 0^oF for 16 hours followed by 24 hours thawing at 140^oF and 2 hours at 77^oF.
- Measure the TS after cycles 1 and 3 for the second and third subset, respectively.
- Calculate the TSR ratio after cycles 1 and 3.

A minimum value of 70 psi at 77^oF for the unconditioned TS and 80% for the TSR after 1 F-T cycle were adopted. Figure 2 summarizes the test results for the TS and TSR for all the field-produced and laboratory-produced mixtures with the error bars representing the 95% confidence interval. Overlapping of the confidence intervals implies the similarity in the measured TS between the mixtures' types. In summary, the data showed that all mixtures met the minimum TS criterion of 70 psi at 77^{o}F after 1 F-T cycle. Additionally, none of the mixtures required anti-strip additives to pass the Superpave moisture sensitivity criterion of 80% TSR after 1 F-T cycle. On the other hand, all mixtures exhibited a TSR value lower than 80% after 3 F-T cycles.

The data in Figure 2 show that in the case of both field-produced and laboratory-produced mixtures the addition of RAP increased both the unconditioned and moisture conditioned TS after 1 and 3 F-T cycles. However, a significant reduction in the TS was observed after 3 F-T cycles for all mixtures. Consequently, the HMA mixtures with RAP did not exhibited an increase in moisture damage due to the use of RAP when compared to virgin mixes.

A paired mean comparison analysis at a significance level of 0.05 was conducted to determine whether there is any statistical significant difference between the TS of field-produced and laboratory-produced mixtures. The following conclusions can be made:

- In general, the TS of the laboratory-produced mixtures were either similar or statistically significantly higher than the TS of the field-produced mixtures. In other words the laboratory-produced mixtures were generally found to be stronger and more durable than the corresponding field-produced mixtures.
- Laboratory-produced mixtures with up to 15% RAP exhibited TS significantly higher than the field-produced mixtures at 0 and 1 F-T cycles while similar TS were observed at 3 F-T cycles.
- Laboratory-produced mixtures with 50% RAP exhibited TS similar to the field-produced mixtures except for the laboratory-mixture with PG58-28 binder at 0 F-T cycles and the laboratory-produced mixture with PG52-34 at 3 F-T cycles which exhibited significantly higher and lower TS that the field-produced mixtures, respectively.

Overall, the test results on the laboratory-produced mixtures can be used to evaluate the relative resistance of the field-produced mixtures to moisture damage. The ranking of the mixtures using the data for laboratory-produced mixtures was similar to the ranking of the field-produced mixtures.

Dynamic Modulus at Multiple F-T Cycles

The AASHTO MEPDG uses the dynamic modulus (|E*|) master curve to evaluate the structural response of the HMA pavement under various combinations of traffic loads, speed, and environmental conditions [7]. The dynamic modulus test was performed according to AASHTO TP62-07 [5] to generate the dynamic modulus master curve of the various mixtures.

All mixtures were evaluated at the unconditioned (i.e. un-damaged condition) and the moisture conditioned (i.e. moisture-damaged condition) stages. The moisture conditioning consisted of subjecting the samples to multiple F-T cycling. A total of three samples from each mix were evaluated following the procedure outlined below:

- Measure the unconditioned |E*| master curve (i.e. 0 F-T cycles).

- Subject the samples to 75% saturation.
- Subject the saturated samples to multiple F-T cycling wherein one F-T cycle consists of freezing at 0°F for 16 hours followed by 24 hours thawing at 140°F and 2 hours at 77°F.
- Subject each sample to the required number of freeze-thaw cycles.
- Conduct |E*| testing after cycles 1 and 3.

All laboratory-produced loose mixtures were subjected to short-term oven aging for four hours at 275°F before compaction while field-produced mixtures were compacted directly. The measured |E*| properties were examined as a function of F-T cycles.

Figure 3a shows the |E*| for the various field-produced and laboratory-produced mixtures for different F-T cycles at 10 Hz loading frequency, representing average highway traffic, and temperature of 77°F. The error bars represents the 95% confidence interval for the average |E*|.

Examining the |E*| data in Figure 3a leads to the observation that the |E*| property of both field- and laboratory-produced mixtures become lower as the mixtures are subjected to multiple F-T cycles. Additionally, at a given F-T cycle, the |E*| property increased with the increase in RAP content. A reduction in the |E*| property was observed for the mixtures with 50% RAP and PG52-34 when compared to the mixtures with 50% RAP and PG58-28.

A paired mean comparison analysis at a significance level of 0.05 was conducted to determine whether there is any statistical significant difference between the |E*| at multiple F-T cycles of field-produced and laboratory-produced mixtures. As a result, the field-produced mixtures exhibited a significantly higher |E*| property than the laboratory-produced mixtures except for the field- and laboratory-produced mixtures with 0% and 50% RAP (without grade change, i.e. with PG58-28) after 1 F-T cycle and the field and laboratory-produced mixtures with 0% and 15% RAP after 3 F-T cycles which exhibited similar |E*| properties. Again, the observed stiffer |E*| in general may be the result of the difference between the field and laboratory aging. This finding is consistent with the binder data presented above which showed stiffer binders for the field-produced mixtures when compared to the laboratory-produced mixtures.

Figure 3b shows the ratios of the moisture conditioned to the unconditioned |E*| after 1 and 3 F-T cycles. Overall, the data indicate that the retained |E*| ratio for each mix decreases with the number of F-T cycles. The data for the field-produced mixtures show that the use of RAP in the mix resulted in a higher |E*| ratio after 1 F-T cycle when compared to the virgin mix (i.e. 0% RAP) indicating improvement in the resistance of the mixtures to moisture damage. On the other hand, except for the F-50%-200 mix, the use of RAP in the mix resulted in a lower |E*| ratio after 3 F-T cycles when compared to the virgin mix. This indicated that RAP mixtures may be more prone to moisture damage than the virgin mix when multiple F-T cycles are used to assess moisture damage of asphalt mixtures. However, it should be noted that RAP mixtures still exhibited a relatively high |E*| value after 3 F-T cycles when compared to the virgin mix.

Overall, similar trends to the field-produced mixtures were observed for the |E*| ratios of the laboratory-produced mixtures. However, the |E*| ratios of the laboratory-produced mixtures at a given F-T cycle were higher than the corresponding |E*| ratios of the field-produced mixtures with the exception of the mixtures with the 50% RAP manufactured with PG52-34, where the opposite was observed. Therefore, the test results on the laboratory-produced mixtures can be used to evaluate the relative resistance of the field-produced mixtures to moisture damage.

The use of the PG52-34 asphalt binder with the mixtures containing 50% RAP (i.e. F-50%-200) improved the resistance of the mixture to moisture damage as measured by the |E*| ratio after 3 F-T cycles. It is important to note that even though the Pen 150-200 and Pen 200-300 were graded as PG58-28 and PG52-34, respectively, their true high and low performance grades were only within 5°C and 2°C, respectively. Consequently, the observed improvement in moisture damage has to do more with the compatibility of the PG52-34 virgin asphalt binder with the RAP binder.

TSRST at Multiple F-T Cycles

The resistances of the various mixtures to thermal cracking were measured using the Thermal Stress Restrained Specimen Test (TSRST) (AASHTO TP10-93). The test cools down a 2”×2”×10” beam specimen at a rate of 10°C/hour while restraining it from contracting. While the beam is being cooled down, tensile stresses are generated due to the ends being restrained. The HMA mixture would fracture as the internally generated stress exceeds its tensile strength. The temperature and stress at which fracture occurs is referred to as “fracture temperature” and “fracture stress”, respectively. The fracture temperature represents the temperature at which the asphalt pavement will develop a transverse crack due to thermal stresses. The fracture stress controls the spacing of the thermal cracks once they occur. It is believed that a higher fracture stress in the TSRST would indicate a longer spacing of the transverse cracks in the field.

The resistance of the mixtures to thermal cracking was measured at the long-term aged, unconditioned (i.e. 0 F-T) and moisture conditioned (i.e. 3 F-T) stages. The aging of the mixtures followed the Superpave recommendation for long-term aging of HMA mixtures which consisted of subjecting the compacted samples to 185°F for 5 days in a forced draft laboratory oven. All compacted samples were long-term aged since low-temperature cracking is a long-term pavement distress mode.

The data in Figure 4 indicate an increase in both unconditioned and moisture conditioned fracture stresses of the field-produced mixtures when 50% RAP was used. In the case of laboratory-produced mixtures, statistically similar fracture stresses were observed at a significance level of 0.05. An average reduction of 53% was observed for the fracture stress after 3 F-T cycles. The fracture temperatures of the mixtures manufactured with PG58-28 asphalt binders decreased with the increase of RAP content. An average reduction of 6°C was observed for the TSRST fracture temperatures at 0 and 3 F-T cycles when 50% RAP was used. On the other hand, mixtures manufactured with the PG52-34 asphalt binder exhibited fracture temperatures that are similar to the virgin mixtures (i.e. 0% RAP). Overall, similar fracture temperatures were observed between field-produced and laboratory-produced mixtures at 0 and 3 F-T cycles. Consequently, laboratory-produced mixtures can be used to evaluate the anticipated relative resistance of field-produced mixtures to thermal cracking based on fracture temperatures.

The majority of the mixtures after 0 and 3 F-T cycles met the target low temperature performance grade of the asphalt binder (i.e. -28°C). The TSRST fracture temperatures of the mixtures with 0% and 15% RAP were within 1°C of the critical low temperatures of the asphalt binders recovered from the same corresponding mixtures (Figure 1). On the other hand, mixtures containing 50% RAP exhibited fracture temperatures that were colder than the critical low temperatures of the asphalt binders recovered from the same corresponding mixtures by 5 to 8°C.

IMPACT OF MOISTURE DAMAGE ON ASPHALT PAVEMENT RESPONSES

The impact of moisture damage on asphalt pavement responses was evaluated through a simplified mechanistic analysis of a typical asphalt pavement. The analyses used the undamaged (i.e. 0 F-T) and moisture-damaged (i.e. 3 F-T) |E*| properties of the various mixtures to evaluate the response parameters of asphalt pavements that are considered critical to fatigue and rutting of the asphalt layer. The following pavement structures were analyzed with the layout shown in Figure 5.

- Asphalt layer: 6 inch thick, modulus varies depending on the type of mix used.
- Crushed aggregate base layer: 10 inch thick, modulus = 20,000 psi.
- Subgrade layer: infinite, modulus = 8,000 psi.

The loading consisted of a single axle load of 18-kips with dual tires at an inflation pressure of 120 psi. The values of the |E*| for the various mixtures were obtained from the dynamic modulus master curves at 0 and 3 F-T cycles for a loading frequency of 10 Hz. |E*| properties at 77°F and 104°F were used for fatigue and rutting analysis, respectively. The temperatures of the |E*| were selected to represent the critical conditions for fatigue cracking of intermediate temperature and permanent deformation of high temperature. Using the four types of asphalt mixtures along with the field and laboratory-produced mixtures resulted in 32 different pavement structures.

The AASHTO MEPDG [8] relates bottom-up fatigue cracking to the tensile strain at the bottom of the asphalt layer and the permanent deformation of the asphalt layer to the vertical compressive strain at the middle of the asphalt layer. The properties of the pavement structures along with the loading conditions were used in the multi-layer elastic solution to calculate the maximum tensile strain (ε_t) at the bottom of the asphalt layer and the maximum vertical compressive strain (ε_v) at the middle of the asphalt layer for all 32 pavements. In the MEPDG, the calculated strains are then input into the performance models of the HMA layer to estimate the fatigue and rutting performance of the HMA pavement. Since, the fatigue and rutting performance models were outside the scope of this study, only pavement responses (i.e. ε_t and ε_v) for the undamaged and moisture-damaged HMA were evaluated and compared to each others. Table 5 summarizes the results of the mechanistic analysis in terms of the calculated strains and the percent increase in strain due to moisture-damaged for all the evaluated pavements.

In general, the data in Table 5 show that the mixtures containing RAP resulted in lower ε_t and ε_v than the virgin mixes at both 0 and 3 F-T. This is due to the higher stiffness (|E*|) for the HMA mixtures with RAP. However, there are three exceptions to the above stated observation: 1) the

F-15%-150 mix exhibited an ε_v after 3 F-T that is slightly larger than the virgin mix 2) the L-15%150 mix exhibited an ε_v after 0 F-T that is larger than the virgin mix and 3) the L-50%-150 mix exhibited an ε_v after 3 F-T that is similar to the virgin mix.

The data highlight the impact of the reduction in stiffness due to moisture damage on pavement responses. Moisture damage resulted in an increase in ε_t and ε_v as high as 40% and 84%, respectively. Such increase in strains would definitely translate into a significant decrease in pavement performance against fatigue and rutting.

By examining ε_t in Table 5, the same relative rankings were observed for field-produced and laboratory-produced mixtures at both 0 and 3 F-T cycles. On the other hand, ε_v at both 0 and 3 F-T cycles did not result in similar relative ranking for the field-produced and laboratory-produced mixtures. Since the mechanistic analysis is mainly driven by the |E*| of the HMA mixture, it is clear that a weak correlation exists between the stiffness of the field-produced mixtures and the laboratory-produced mixtures at 104°F.

FIELD PERFORMANCE

In October 2010, a condition survey of the test sections was performed by the Western Research Institute (WRI) personnel. Figure 6 shows pictures for all four test sections taken at the time of assessment. At that time, the inspection revealed no distresses in the 0, 15 and 50% RAP pavements after 13 months of service. Thermal cracking and moisture damage were not apparent at either section. Overall, the pavement condition was excellent and uniformly the same along the total length of all test sections. Field performance will continue to be monitored over the next few years and pavement distress survey and falling weight deflectometer (FWD) data will be collected.

FINDINGS AND CONCLUSIONS

This research effort conducted an extensive laboratory evaluation of field-produced and laboratory-produced mixtures with RAP content up to 50%. The impact of high RAP content on moisture damage and thermal cracking resistance of HMA mixtures were evaluated using advanced testing techniques. No change to the virgin binder grade was deemed necessary for mixtures with 15% RAP. However, the blended asphalt binder from 50% RAP mixtures failed to meet the low target PG of -28°C even when a PG52-34 was used. The following summarizes the findings based on this study for the evaluated mixtures:

- In general, the use of multiple F-T cycles provided a better characterization of the mixtures resistance to moisture damage.
- The AASHTO T283 test at multiple F-T cycles did not show additional reduction in the resistance of the HMA mixtures to moisture damage because of the use of 50% RAP.
- In general, higher or similar TS were observed for the laboratory-produced mixtures when compared to the field-produced mixtures.

- The |E*| test after 3 F-T cycles showed additional reduction in the resistance of the HMA mixtures to moisture damage because of the use of 50% RAP. However, RAP mixtures still exhibited a relatively high |E*| after 3 F-T cycles when compared to the virgin mix.
- No additional reduction in the TSRST fracture stress after 3 F-T cycles was observed because of the use of RAP.
- The use of 50% RAP without a grade change for the virgin binder resulted in a reduction in the TSRST fracture temperature. However, the use of a softer virgin binder with the 50% RAP mixture resulted in a similar fracture temperature to that of the virgin mix.
- The TSRST fracture temperatures of the evaluated mixtures with 0% and 15% RAP were within 1°C of the recovered asphalt binders' critical low temperatures. On the other hand, the TSRST fracture temperatures of mixtures with 50% RAP were colder than recovered asphalt binders' critical low temperatures by 5 to 8°C.
- Overall, field-produced and laboratory-produced mixtures ranked similarly in the AASHTO T283, |E*| and TSRST tests.

In summary, the HMA mixtures with 50% RAP resulted in acceptable resistance to moisture damage with a better resistance for the mixture with PG52-34 (i.e. Pen 200-300) asphalt binder. The observed difference in the mixtures' resistance to moisture damage has to do more with the compatibility of the PG52-34 virgin asphalt binder with the RAP binder. The mixtures with 50% RAP exhibited an acceptable resistance to thermal cracking as measured with the TSRST with, again, a better resistance for the mixture with PG52-34 asphalt binder. The monitoring of the field performance will help validating the findings of this study and hopefully explain the difference between the TSRST fracture temperatures and the recovered asphalt binders' critical low temperatures of the 50% RAP mixes.

Regardless of the RAP content, the Superpave procedure of 4 hours at 275°F in a forced draft oven did not simulate the aging of the evaluated field-produced mixtures.

Overall, all test results showed that laboratory-produced mixtures can be used to evaluate the relative resistance of the field-produced mixtures to moisture damage and thermal cracking. However, some differences in the measured values were observed between the field-produced and laboratory-produced mixtures which may require adjustment to any criteria used.

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TABLE 1 Traffic Information.

Property	Values
Two-Directional average daily traffic (ADT)	25,395
Truck percent of ADT, %	7.6
Two-Directional average daily truck traffic (ADTT)	1,930
Directional split, %	50/50
Annual growth rate, %	2.0
Annual one-directional design ESAL	80,000
20 year one-directional design ESAL	1,950,000

TABLE 2 Experimental Program.

Property*	Mixture ID							
	F-0%-150	F-15%-150	F-50%-150	F-50%-200	L-0%-150	L-15%-150	L-50%-150	L-50%-200
AASHTO T283 at multiple F-T:								
• TS vs. F-T cycles: 0, 1 and 3 F-T	X	X	X	X	X	X	X	X
• TSR at 1 and 3 F-T	X	X	X	X	X	X	X	X
Mechanical property								
• |E*| vs. F-T cycles: 0, 1 and 3 F-T	X	X	X	X	X	X	X	X
Resistance to thermal cracking:								
• TSRST: 0 and 3 F-T	X	X	X	X	X	X	X	X

* TS denotes tensile strength, F-T denotes freeze-thaw, TSR denotes tensile strength ratio, |E*| refer to dynamic modulus, and TSRST denotes thermal stress restrained specimen test.

TABLE 3 Gradations of Extracted Aggregates.

Sieve Size	Mixture ID							
	F-0%-150		F-15%-150		F-50%-150		F-50%-200	
	% Passing	JMF	% Passing	JMF	% Passing	JMF	% Passing	JMF
3/4"	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
1/2"	93.0	90-93	90.5	89-93	93.1	90-93	93.0	90-93
3/8"	84.0	79-84	81.9	78-83	85.3	78-84	85.1	78-84
#4	64.0	57-64	61.7	57-64	66.3	59-66	65.6	59-66
#8	51.7	-	49.9	-	54.4	-	53.4	-
#10	49.0	43-50	46.2	43-50	50.3	45-51	48.4	45-51
#16	42.6	-	40.3	-	42.9	-	42.5	-
#40	28.1	24-31	25.8	24-30	25.2	23-28	25.9	23-28
#50	18.5	-	15.6	-	15.6	-	16.5	-
#100	4.8	-	4.3	-	5.5	-	5.6	-
#200	4.2	4-5	4.6	4-6	5.0	4-6	5.2	4-6

TABLE 4 Superpave PG Grades of Various Asphalt Binders.

Binder	PG Grade
Pen 150-200	58-28
Pen 200-300	52-34
RAP Binder	76-10
Recovered from F-0%-150	58-28
Recovered from F-15%-150	58-28
Recovered from F-50%-150	64-16
Recovered from F-50%-200	64-22
Recovered from L-0%-150	58-28
Recovered from L-15%-150	58-28
Recovered from L-50%-150	64-22
Recovered from L-50%-200	58-22

TABLE 5 Results of the Mechanistic Analysis.

Mixture ID	0 F-T		3 F-T		Percent increase in ε_t	Percent increase in ε_v
	Tensile strain at 77°F in the middle at the bottom of the HMA layer, ε_t (microns)	Vertical compressive strain at 104°F in the middle of the HMA layer, ε_v (microns)	Tensile strain at 77°F in the middle at the bottom of the HMA layer, ε_t (microns)	Vertical compressive strain at 104°F in the middle of the HMA layer, ε_v (microns)		
F-0%-150	268	651	338	738	26%	13%
F-15%-150	239	474	317	747	33%	58%
F-50%-150	173	626	242	538	40%	-14%
F-50%-200	199	375	216	454	9%	21%
L-0%-150	297	870	343	1020	15%	17%
L-15%-150	283	931	323	761	14%	-18%
L-50%-150	214	555	268	1020	25%	84%
L-50%-200	268	854	298	886	11%	4%

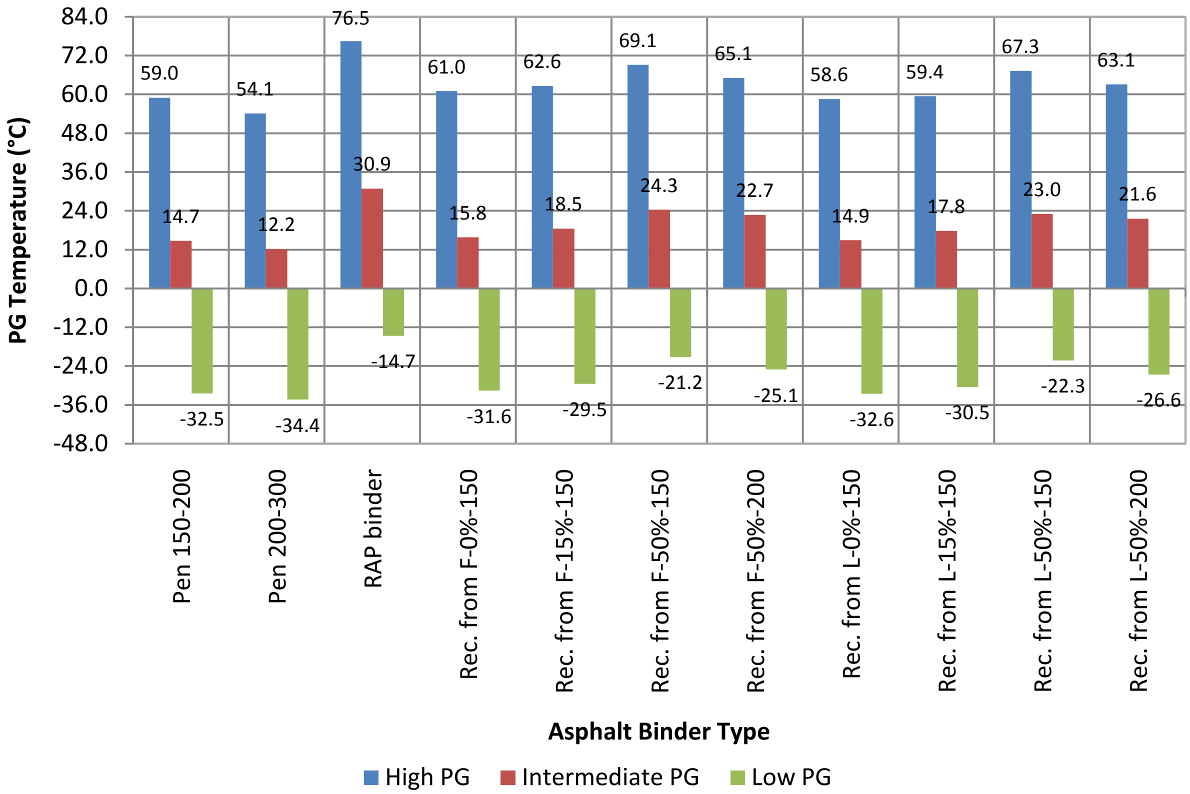
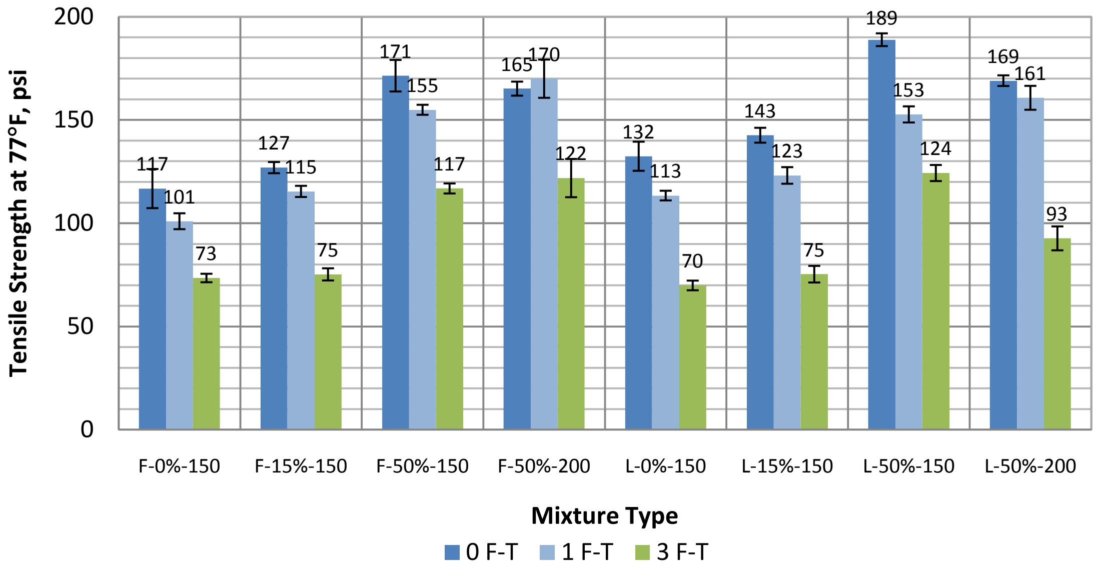
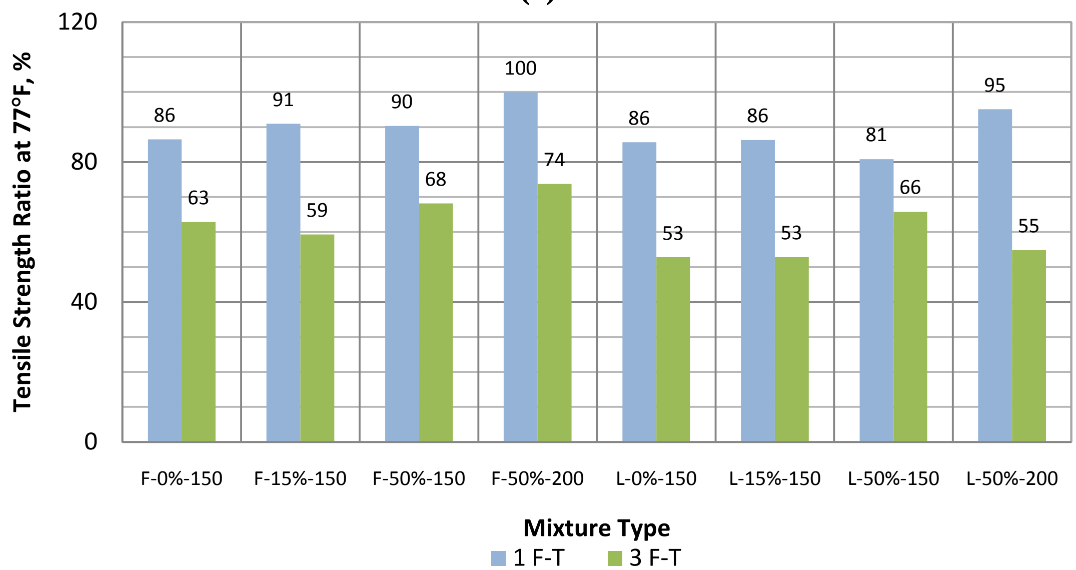


FIGURE 1 Superpave PG Temperatures of Various Asphalt Binders

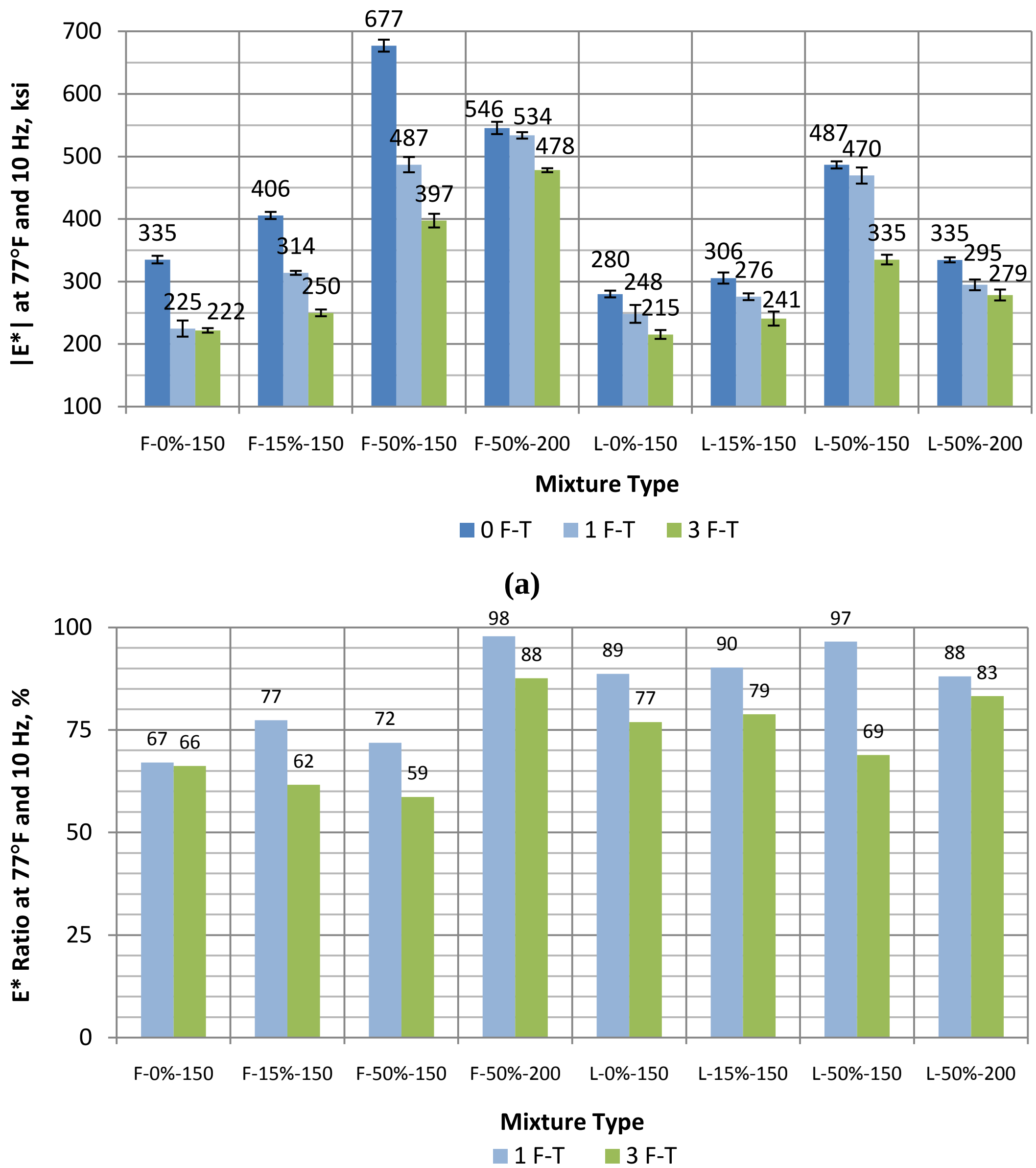


(a)



(b)

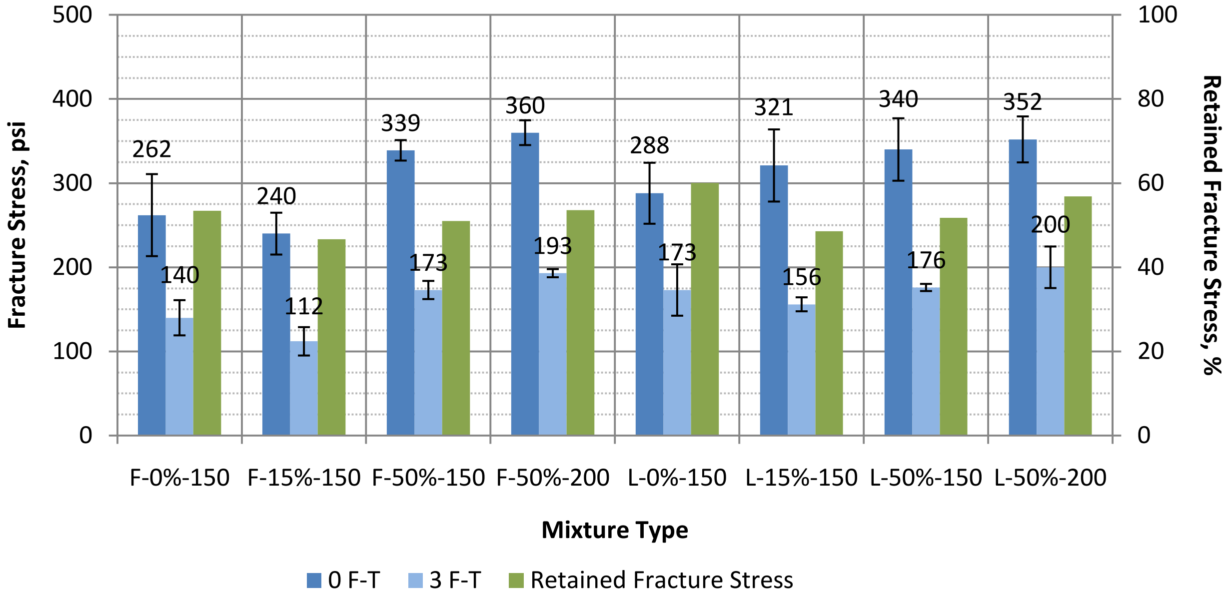
FIGURE 2 (a) Tensile strength values at 77°F and 0, 1 and 3 F-T (b) TSR values at 1 and 3 F-T. (Numbers above bars represent mean values and whiskers represent mean ± 95% confidence interval)



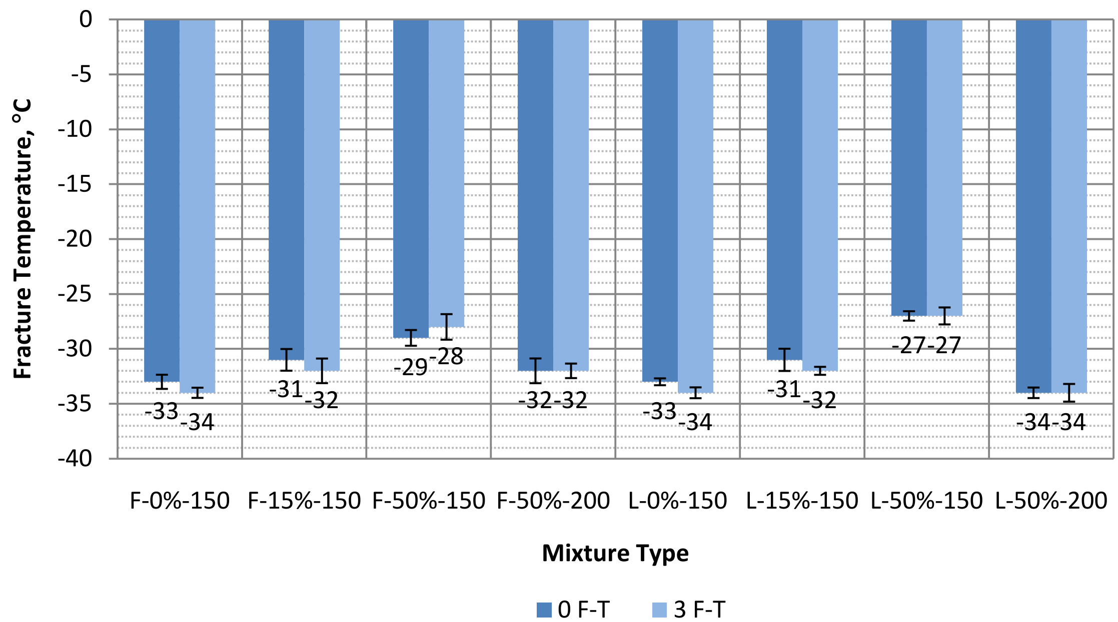
(a)

(b)

FIGURE 3 (a) |E*| at 77°F as a function of F-T cycles (b) |E*| ratio at 1 and 3 F-T cycles. (Numbers above bars represent mean values and whiskers represent mean ± 95% confidence interval)



(a)



(b)

FIGURE 4 Thermal cracking characteristics of the various mixtures (a) fracture stresses at 0 and 3 F-T cycles (b) fracture temperatures at 0 and 3 F-T cycles. (Numbers above bars represent mean values and whiskers represent mean ± 95% confidence interval)

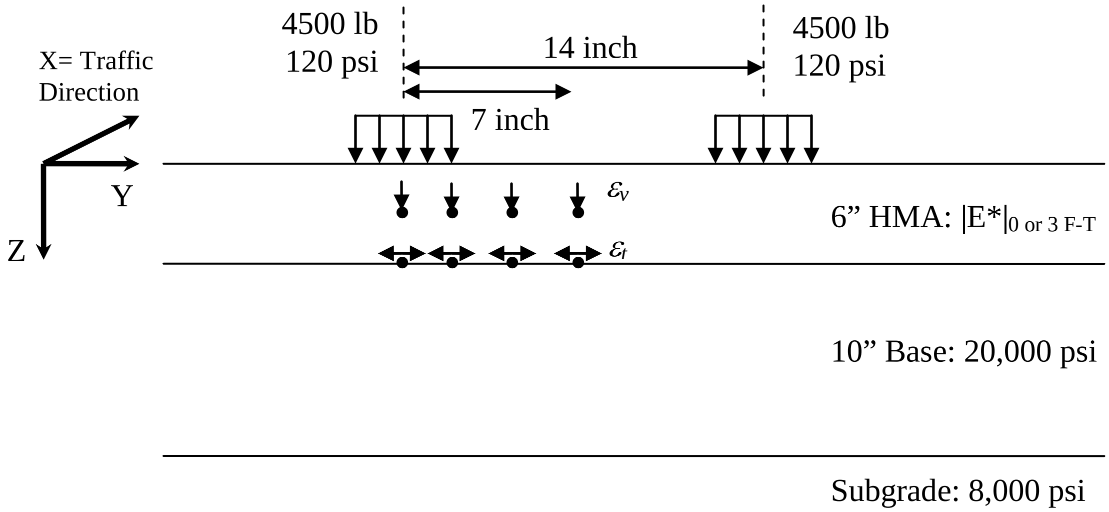


FIGURE 5 Pavement structure layouts for mechanistic analysis

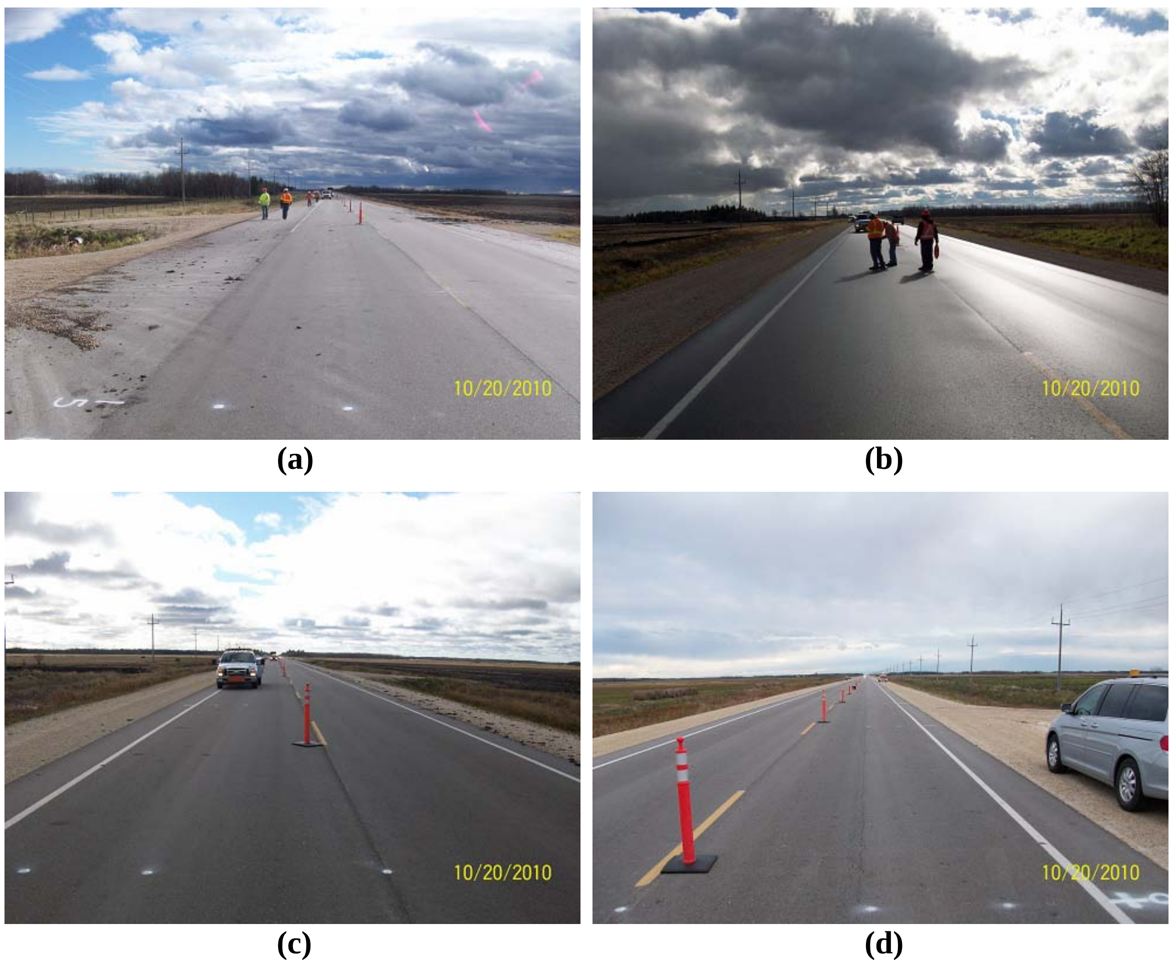


FIGURE 6 Test sections field performance (a) 0% RAP (b) 15% RAP (c) 50% RAP no grade change (d) 50% RAP with grade change