Influence of Bond Conditions on the Performance of Ultra-Thin Whitetopping (Bonded Concrete Resurfacing)

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ABSTRACT

Ultra-Thin Whitetopping (UTW) is a pavement rehabilitation technique that involves the placement of a thin Portland Cement Concrete (PCC) overlay, 50 mm to 100 mm thick, over a distressed bituminous pavement. Performance of the UTW is dependent on the bond between the two layers. This study evaluated the influence of bond condition on the performance of UTW pavements. Two different test areas were constructed in the INDOT/Purdue University Accelerated Pavement Testing facility. Each test area had different lanes that varied by concrete mix design, bond preparation, and pavement cross-section. These lanes were subjected to approximately 300,000 load applications. The pavements were instrumented and strains were recorded under loading. Evaluations were performed periodically to monitor distress development.

With respect to the interfacial bond conditions, the observed amount of distress generally followed the expected trends. Lanes with a lower degree of bonding (partially bonded or unbonded conditions) typically exhibited higher amounts of distress. Conversely, fully bonded lanes typically showed the lowest degree of distress.

The interfacial bond conditions did affect the strain distribution and maximum load-induced strains. The measured strains in the bonded lanes matched the theoretical values well. There was some discrepancy between the measured and theoretical unbonded strain distributions. Further analysis into this discrepancy indicated that even in an unbonded condition there was a moderate level of friction between the UTW and Hot Mix Asphalt (HMA) layers.

The interfacial bond conditions did have an influence on the estimated radius of relative stiffness as determined by Falling Weight Deflectometer (FWD). The results indicate that the FWD can be used as a method to determine whether or not the UTW was bonded. However, using the FWD to determine the degree of bonding did not provide very good results. The relative values of the radius of relative stiffness matched the trend exhibited by the bond strengths for cores taken from the same lanes.

INTRODUCTION

Ultra-thin whitetopping (UTW) is a pavement rehabilitation technique that involves the placement of a thin Portland Cement Concrete (PCC) overlay, 50 mm to 100 mm thick, over a distressed bituminous pavement. Typically, the bituminous pavement is milled and cleaned which helps to create a bond between the existing bituminous pavement and the UTW

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overlay. The bond between the two layers promotes composite action of the pavement section. The composite action between the two layers permits a reduction in the thickness of the UTW layer. An additional reduction in stress is also achieved by using a short joint spacing. It is thought that the short joint spacing causes the slab to deflect rather than bend under load, Mack et al. [1998].

While it is generally agreed that the deterioration of the bond will cause the pavement to fail, little investigation has been done into how this bond degrades under loading. Armaghani and Tu [1997] in their study of Florida UTW sections noted that when debonding of the UTW layer occurs, cracking of the UTW layer soon follows. Addressing the development of cracks in the UTW at MnROAD, Burnham [2005] also indicated that debonding occurred before cracking in the UTW pavement. However, Rasmussen et al. [2002] in their review of the FHWA ALF results proposed that corner cracking was due to the local densification of the Hot Mix Asphalt (HMA) at the corner location. The name given to this phenomenon was "virtual void". As the UTW performance is based on the composite action with the underlying HMA, when debonding of the UTW layer occurs, the strains in the UTW layer would increase. It would thus be expected that the sections with lower bond strengths (partially bonded or debonded conditions) would exhibit greater distress than fully bonded sections. To evaluate these effects, three different bond preparation techniques were employed in this study which resulted in three general interfacial bond conditions: fully bonded, partially bonded, and debonded.

Traditionally UTW has been used to rehabilitate HMA pavements. However, Cable et al. [2001] noted the need to consider the use of UTW over composite pavements. Rajan and Olek [2001] also indicated that the performance of UTW is different when the UTW is constructed over a composite pavement (bituminous pavement over portland cement concrete pavement) than when constructed over an HMA pavement. In the NCHRP report on the state of the practice of UTW, Rasmussen and Rozycki [2004] noted that one major factor influencing distress development in UTW pavements was the quality of the underlying layers. Sections constructed with HMA mixes that had a higher quality; that is mixes that were more rut resistant, performed better. Thus, it is hypothesized that the stiffness of all layers underlying the UTW will affect distress development in the UTW layer whether the underlying section consists of an existing HMA or composite pavement. As such, two underlying pavement sections were varied in this study: a composite section and an HMA section.

According to the PCA fatigue equations, Wu et al. [1998], if a higher strength concrete is used in the UTW, the number of load applications it can withstand before cracking is also higher. In general, the concrete used in UTW applications is a better quality than that used in typical paving applications. However, the addition of fibers to the mix as well as high early strength concrete mixtures has been proposed to improve the performance of the UTW. Thus, these concrete constituents are also varied in four different mix designs: base mix, fiber mix, high-early strength mix, and high-early strength mix with fibers.

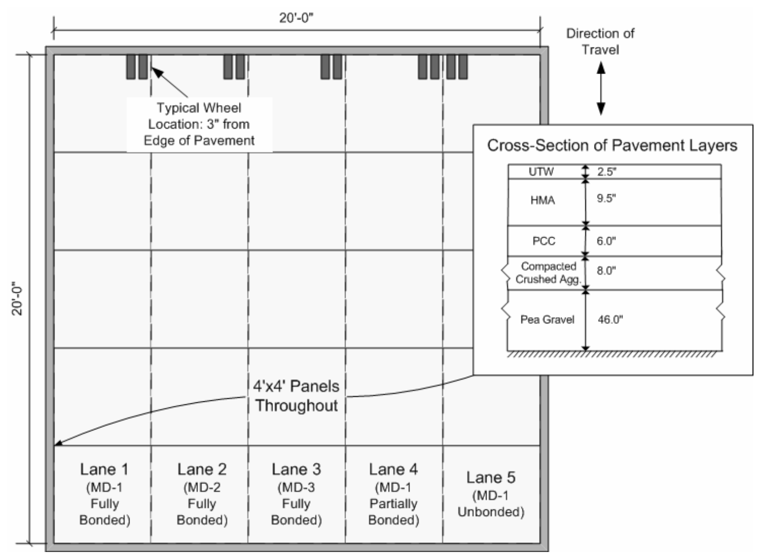
RESEARCH SIGNIFICANCE

The performance of UTW is highly dependent upon the bond between the UTW and bituminous pavement layers. This paper evaluates the effect of three different bond conditions on the load-induced strains, distresses, and radius of relative stiffness.

EXPERIMENTAL EVALUATION

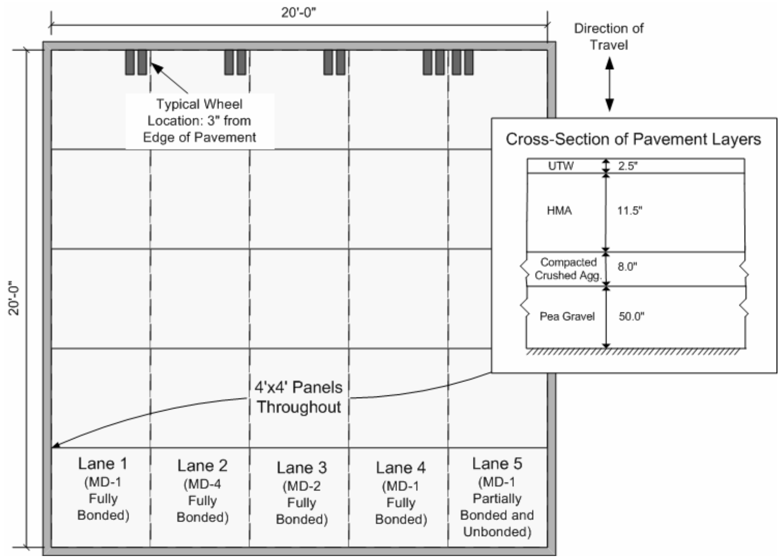
Test Areas

Two different UTW pavement sections were constructed in an accelerated pavement testing facility. These sections varied primarily in the design of their underlying pavement layers. In Test Area 2 (TA2) the UTW was placed over a more realistic composite section consisting of HMA over a thinner plain concrete slab. Test Area 3 (TA3) was constructed with UTW over an existing HMA pavement (no underlying concrete slab). TA2 and TA3 had five test lanes. Individual test lanes within TA2 and TA3 were constructed with one of four different concrete mix designs: MD-1 was a base concrete mix; MD-2 was a high early strength mix; MD-3 was a fiber reinforced mix; and MD-4 was a high early strength mix with fibers. TA2 and TA3 also had test lanes that varied by the bond preparation done in the lanes. An overview of these test areas is contained in Figure 1.



(a)

FIGURE 1 Layout of Test Areas (a) TA2 (b) TA3



(b)

FIGURE 1 (continued) Layout of Test Areas (a) TA2 (b) TA3
(Note: 1 inch (1”) = 25.4mm; 1 foot (1’) = 305mm)

Accelerated Pavement Testing Facility

Testing was conducted at the Indiana Department of Transportation (INDOT) – Purdue University Accelerated Pavement Testing (APT) Facility located at the INDOT Office of Research and Development in West Lafayette, IN. The APT is housed in approximately 2000 ft^2 (185 m^2) environmentally controlled building. The APT facility is composed of test pit, a loading mechanism and control and monitoring equipment.

The APT testing area consists of a 20 foot (6100 mm) wide by 20 foot (6100 mm) long by 6 foot (1800 mm) deep test pit. The pit allows the placement of full depth pavements. The operator control room houses both the software and hardware used to fully control the APT operation. The room is currently equipped with three personal computers networked together. The first computer is used to fully control and operate the APT, the second computer is used for data collection/reduction and the third computer is used for monitoring and diagnoses of the APT hardware.

The APT carriage, which can be equipped with either a full-size, dual-tire truck wheel or a super-single, half-axle assembly, is cable driven by a 30 hp (22 kW) motor and a 50 hp (37 kW) drive. This setup allows speeds up to 5 mph (8 km/h). The APT loading mechanism uses a system of four air pistons that maintain a constant force up to 20,000 lbs (89 kN). This load can be programmed either in static or dynamic modes. These cylinders also allow the wheels to be raised and returned to the start-up position of the test pad in one direction type of testing.

Construction of UTW Test Areas

For the construction of the pavement in TA2 and TA3, the APT test pit was completely emptied and cleaned. In both test areas, the test pit was filled with 46 to 48 inches (1150 to 1200 mm) of pea gravel and an 8 inch (200 mm) layer of dense graded crushed limestone was placed over the pea gravel. In TA2, a 6 inch (150 mm) thick plain PCC layer was placed over the crushed stone. The PCC layer was then overlaid by four HMA layers totaling 12 inches (300 mm). The HMA layers were Superpave intermediate and surface mixtures with PG 64-28 binder. 2.5 inches (63 mm) of the HMA layer were milled to allow the construction of the UTW layer. This left the HMA layer approximately 9.5 inches (237 mm) thick. In TA3, the, six HMA layers (PG 64-28; base, intermediate, and surface mixtures) totaling approximately 14 inches (350 mm) thick were placed over the crushed stone. Again, 2.5 inches (63 mm) of the HMA layer were milled to allow the construction of the UTW layer. This left the HMA layer approximately 11.5 inches (288 mm) thick.

Four different UTW concrete mixtures containing Class C fly ash, with a target water-cementitious material ratio of 0.36, were used for the UTW in this experiment. The UTW concrete mixture designs are presented in Table 1.

In TA2 and TA3, five different UTW test lanes were constructed. Each lane was constructed after the APT loading of the previous lane was completed. In order to evaluate different bond interfaces between the HMA and UTW, three different surface preparation techniques were employed. The surface of the milled HMA was swept and cleaned with a compressed air hose prior to UTW placement in Lanes 1, 2, and 3. In Lane 4, the milled HMA surface was not cleaned prior to placement of the UTW in TA2, but was cleaned as in the other lanes for TA3. A sheet of plastic was placed over the milled HMA layer in Lane 5: full lane in TA2 and half of the lane in TA3. The remaining portion of Lane 5 was not cleaned prior to UTW placement.

TABLE 1 Mix Design Proportions

Material	**lb/cyd (kg/m^3)**
Cement (Type I – MD-1 and MD-3; Type III – MD-2 and MD-4)	635 (375)
Fly Ash (Class C)	118 (70)
Coarse Aggregate (Limestone - INDOT #11)	1695 (1000)
Fine Aggregate (Natural – INDOT #23)	1271 (750)
Water	271 (160)
HRWR	(3-4 l/m^3)
Polypropylene Fibers (20 mm monofilament) (MD-2 and MD-4)	1.8 (1.1)

In all cases the UTW was placed at a width of 4 feet (1.2 m). Separate pours were conducted for each lane. Concrete mixture MD-1 was used in Lanes 1, 4, and 5 in TA2 and TA3. MD-2 was used in Lane 2 and Lane 3 in TA2 and TA3, respectively. MD-3 was used in Lane 3 on TA2. MD-4 was used on Lane 2 in TA3. The UTW was placed and finished by hand. After 24 hours the concrete joints were sawed at a spacing of 48 inches by 48 inches (1200 mm by 1200 mm) for TA2 using a standard pavement saw. In TA3, and early entry saw was used and the joints were cut 48 inches by 48 inches (1200 mm by 1200 mm) within 4 hours after the UTW concrete was placed. For each separate lane, the concrete was covered with wet burlap and plastic and cured for 3 days. Thus, all lanes had cured the same duration prior to loading with the APT.

Accelerated Pavement Testing

The pavement in TA2 and TA3 was tested with a load of 15,000 lbs (67 kN) after the 3-day cure period. For all tests, a single-axle, dual-tire configuration was used and the tire was run bi-directionally over the pavement. Temperature in the APT facility was held relatively constant between 75°F to 85°F (24 ºC to 29 ºC) during testing. For TA2 and TA3, approximately 300,000 load applications were applied to each lane except TA2 Lane 5 which only had 180,000 load applications prior to terminating the test.

Distress Evaluation

Distresses in the pavement were recorded periodically during the APT loading. Crack locations, width, and length were charted. Sounding of the UTW was performed to evaluate the bond. Sounding was done with a hammer in a grid pattern to locate the areas of potential debonding. Limits of the debonding were determined with additional sounding of the suspect areas. Limits were marked with paint on the pavement to assess changes in the debonded areas. Finally, faulting of the joints and cracks was also recorded.

Instrumentation

In order to evaluate the strains throughout the depth of the pavement strain gages were placed in the PCC, HMA, and UTW layers. Three different types of strain gages were used in this experiment. The first gage type, Texas Measurements embedment-type gages (PML-60-2L), was embedded in the UTW and PCC layers at a level of 0.5-inches (12 mm) below the surface and at 0.5-inches (12 mm) from the bottom of the layer. The second gage type, Texas Measurements surface-mount-type gages (WFLM-60-11-2LT), was glued to the surface of the milled HMA. The third type of gage was a CTL (ASG-152) HMA embedment gage that was placed at the bottom of the HMA layer in TA2 and TA3. These gages were connected to a Vishay System 6000 data acquisition system. This system has the ability to take individual strain readings at a very high sampling rate. The high frequency of data sampling enabled the recording of the dynamic strains as the wheel passed over the pavement. Dynamic strain data were collected approximately every 2500 load applications.

Pavement Deflection Analysis

Falling Weight Deflectometer (FWD) data were collected prior to milling of the HMA and before APT loading of the UTW. FWD tests were conducted on each of the five UTW lanes and at four different load levels: 7,000, 9000, 11,000, and 14,000 lbs (3200, 4100, 5000, and 6400 kg) using a Dynatest Falling Weight Deflectometer. Sensors were spaced at -12, 0, 8, 12, 24, 36, 48, 60, and 72 inches (-300, 0, 200, 300, 600, 900, 1200, 1500, and 1800 mm). Tests were taken every 12 inches (300 mm) at nine different locations within a lane. Test data were averaged for each lane. Radius of relative stiffness values were calculated from these data using the methodology included in the AASHTO Design Guide Supplement [1998].

RESULTS

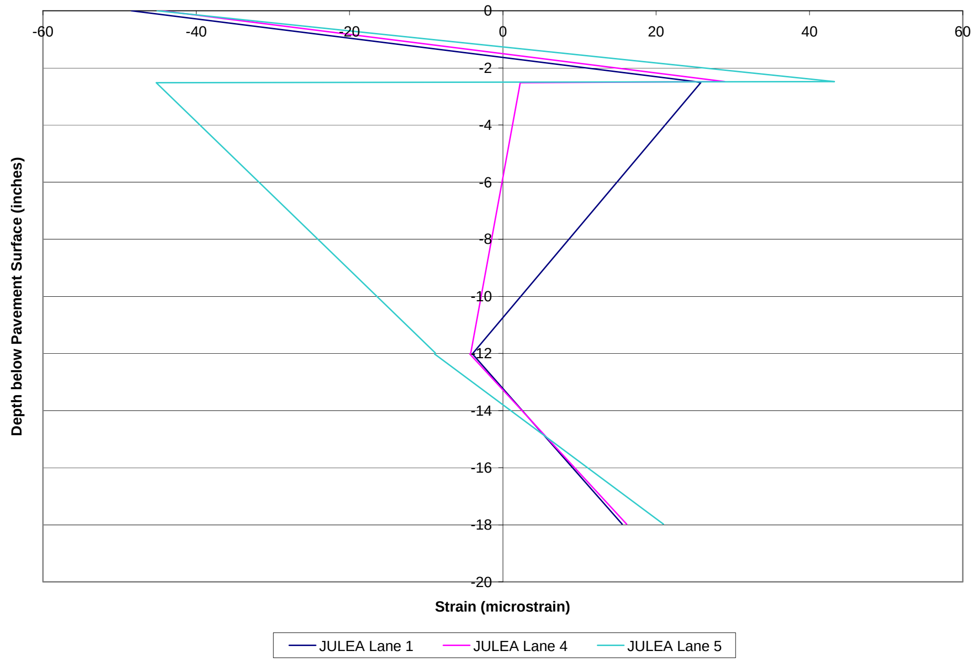
Influence of Interfacial Bond Conditions on Strain Distributions and Maximum Load-Induced Strains

Mack et al. [1998] indicated that the excellent performance of UTW was dependent on the bond between the UTW and HMA layers. The bond allows the two layers to act monolithically and causes the neutral axis to shift down in the pavement, thereby reducing the stress in the concrete below the tensile strength of the concrete.

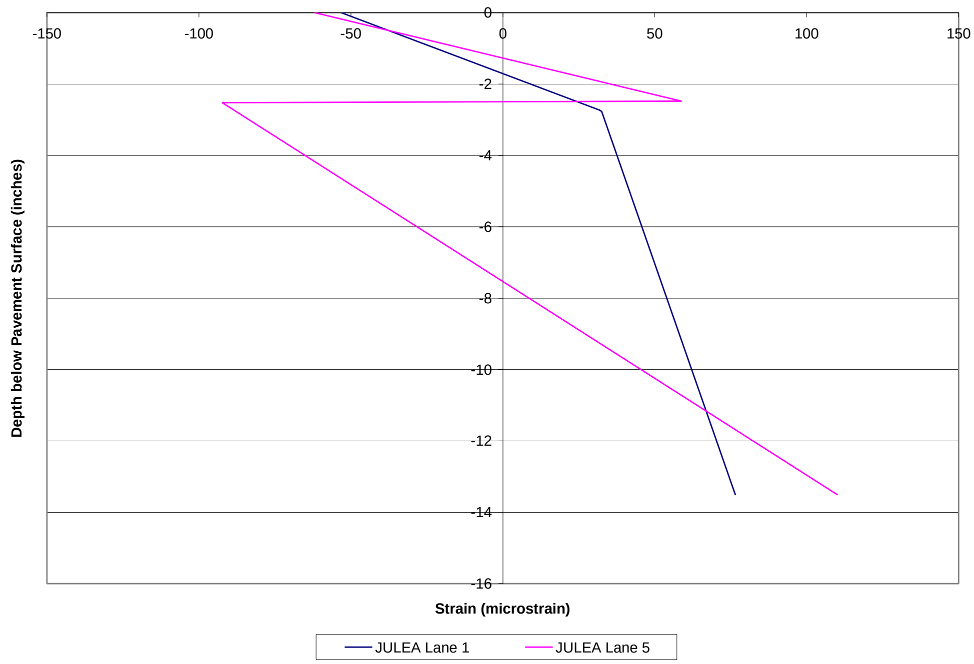
The magnitude of strains in the UTW and HMA layers may also depend on the degree of bond between the layers. To confirm this hypothesis, the layer elastic analysis program WinJULEA was used to calculate the stresses across the pavement depth for Lane 1 in each of the test areas. The strains calculated using this program will heretofore be referred to as "JULEA" strains. Where available, measured values of the layer properties and thicknesses were used as inputs into the program otherwise, the parameters were estimated using standard reported material property values and design thicknesses. A load equal to 15,000 lbs (67 kN) was used as an input for TA2 and TA3 to correspond to the load applied by the APT load carriage. The strains were calculated just above and below the interfaces between the pavement layers and these strains were plotted across the depth of the section. The strains used in the comparison were from Lanes 1 (fully bonded for purposes of this comparison, E=4,440,000 psi (30.6 GPa)), 4 (partially bonded, E=5,640,000 psi (38.9 GPa)), and 5 (debonded, E=5,410,000 psi (37.3 GPa)) in TA2 and Lanes 1 (bonded, 6,910,000 psi (47.7 GPa)) and 5 (debonded, E=6,970,000 psi (46.8 GPa)) in TA3. (Lane 4 for TA3 was not included as it was originally intended to be partially bonded but was inadvertently constructed as fully bonded.) (Moduli for HMA and PCC were 450,000 psi (2.79 GPa) and 5,010,000 psi (34.5 GPa), respectively.) The Win JULEA program had the ability to vary the slippage between the two layers. Thus, for the bond between the UTW and HMA no slippage (a full bond) was input for Lane 1, a moderate degree of slippage (partial bond) was input for Lane 4 in TA2, and full slippage (no bond) was input for Lanes 5 in both TA2 and TA3. The theoretical strain distributions that resulted from these analyses are presented in Figure 2.

As can be seen in Figure 2, the strain distribution and maximum strain magnitudes vary significantly when the interfacial bond conditions are modified for both TA2 and TA3. Also, the neutral axis in the UTW layer shifts higher in the layer when the bond is decreased. This shift results in a corresponding increase in the maximum strain at the bottom of the UTW layer.

The measured strains were filtered to identify the maximum strains under the applied load for the gages places across the pavement sections. For each test area, data from the Lane 1, edge location collected during the "initial" load application (0 to 20,000 load applications) were used for comparison with the theoretical strains calculated for the same location and load conditions. The "initial" loading was chosen as so that the strains used in the comparison would not reflect any fatigue effects from multiple loadings since the layer elastic analysis did not take fatigue effects into account. These data are presented in Figure 3.



(a)



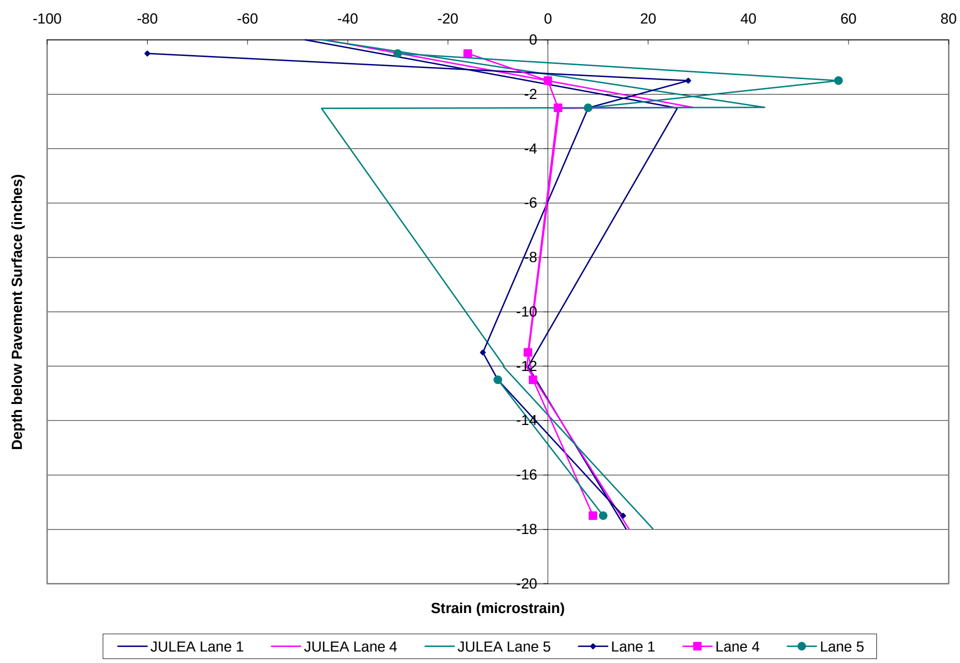
(b)

FIGURE 2 Comparisons of Calculated Strains for Different Bond Conditions (a) TA2 (b) TA3
(Note: 1 inch = 25.4mm)

The strain distribution for the TA2 Lane 1 generally followed the shape of the theoretical distribution but did not match the corresponding strain magnitudes. This discrepancy is particularly evident when the strains are extrapolated to the extremes of the UTW layer (see Table 2). The partially bonded lane in TA2 (Lane 4) did, in general, match the theoretical distribution and magnitudes fairly well. The one exception was a discrepancy between the measured and theoretical strain at the top of the HMA layer. It was thought that this discrepancy was the result of the input parameter used for the modeling of the bond, however, in subsequent runs of the program using different factors the theoretical result presented was the closest match to the measured values.

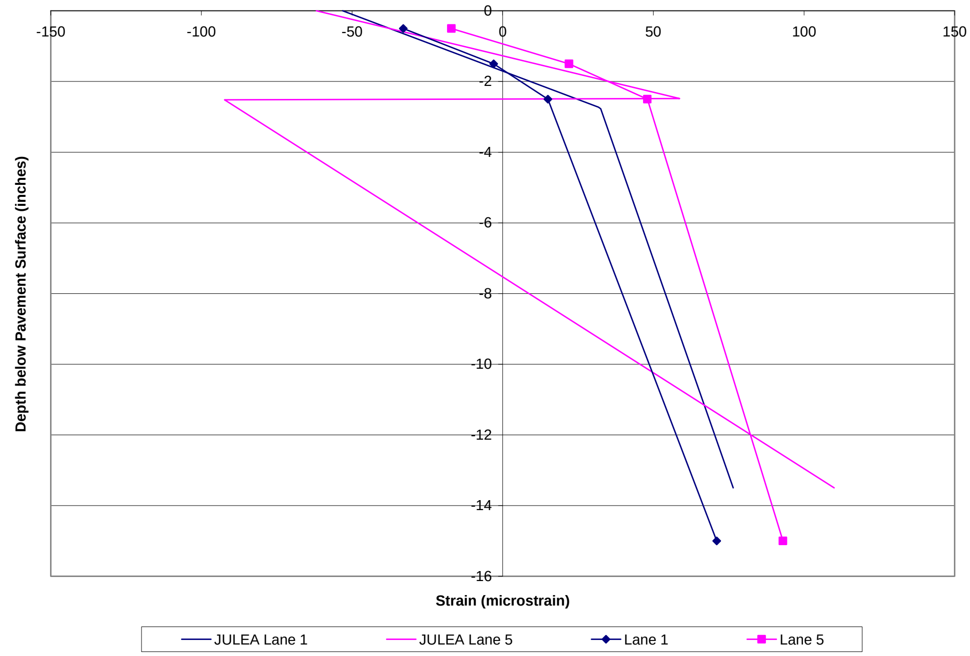
The unbonded lane in TA2 (Lane 5) did not follow the theoretical distribution or match the magnitude of the expected strains in the UTW or HMA layers (though they do match well in the PCC layer). The magnitude of the measured UTW strains was significantly higher than the theoretical strains. Also the measured HMA strain at the top of the layer was actually tensile (positive) while the theoretical strain was negative. This may have been the result of the value of the input used for the bond modeling in the WinJULEA program. As noted previously, the input used for the unbonded condition assumed full slippage between the two layers. This assumption appears to have overestimated the bond condition for an unbonded lane. The measured values indicate that there was at least a moderate level of friction between the two layers.

For test area TA3, the measured strain distribution for Lane 1 (bonded) matched the shape of the theoretical strain distribution and magnitudes fairly well. For Lane 5 (unbonded) however, the strains did not match the expected distribution or magnitudes. This discrepancy was likely the result of the same reasons noted above for Lane 5 in TA2.



(a)

FIGURE 3 Comparisons of Calculated and Measured Strains for Different Bond Conditions (a) TA2 (b) TA3



(b)

FIGURE 3 (continued) Comparisons of Calculated and Measured Strains for Different Bond Conditions (a) TA2 (b) TA3
(Note: 1 inch = 25.4mm)

TABLE 2 Comparisons of Theoretical and Maximum Strains

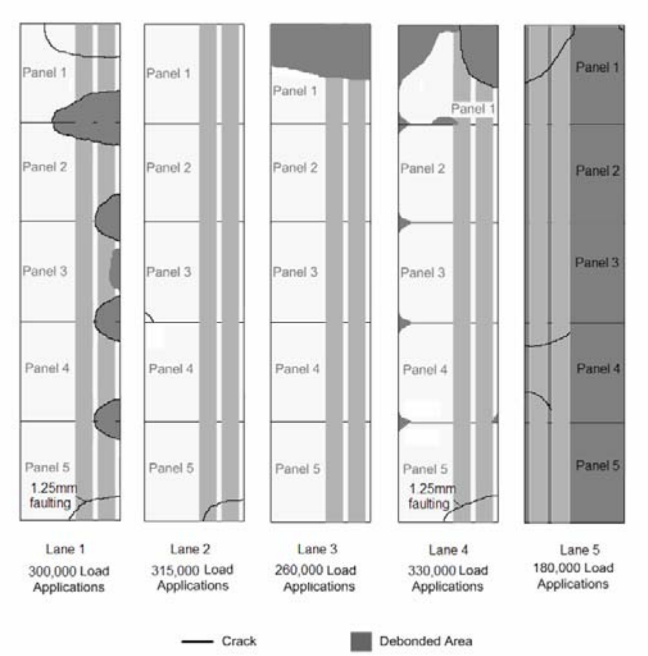
Location	Theoretical Strains (microstrain)	Measured Strains (microstrain)
TA2		
Top UTW	-38	-100
Bottom UTW	23	100
Top HMA	23	8
Bottom HMA	-3.2	-15
Top PCC	-3.2	-15
Bottom PCC	15	16
TA3		
Top UTW	-53	-60
Bottom UTW	32	15
Top HMA	32	15
Bottom HMA	82	75

In regards to the shifting of the neutral axis in the UTW layer, taking into account the discrepancies discussed above, the neutral axes in all test lanes shifted in a similar way as the neutral axes from the theoretical analyses. For those lanes that were bonded well (e.g. TA3 Lane 1) the measured distributions had neutral axes lower in the UTW layer than those that were unbonded (e.g. TA3 Lane 5). Accordingly, the measured strains at the bottom of the UTW layer were less for the fully bonded lanes (e.g. TA3 Lane 1) than in the unbonded lanes (e.g. TA3 Lane 5).

Influence of Interfacial Bond Conditions on Type and Severity of Distresses
Armaghani and Tu [1997] in their study of the Florida UTW sections noted that when debonding of the UTW layer occurs, cracking of the UTW layer soon follows. This finding was also confirmed by Burnham [2005] in his review of the UTW sections installed at MnROAD. During the APT testing it was observed that when debonding of the UTW layer occurred during the APT evaluations, the strains in the UTW layer increase. It was thus expected that the sections with lower bond strengths (partially bonded or debonded) would exhibit greater distress than full bonded sections.

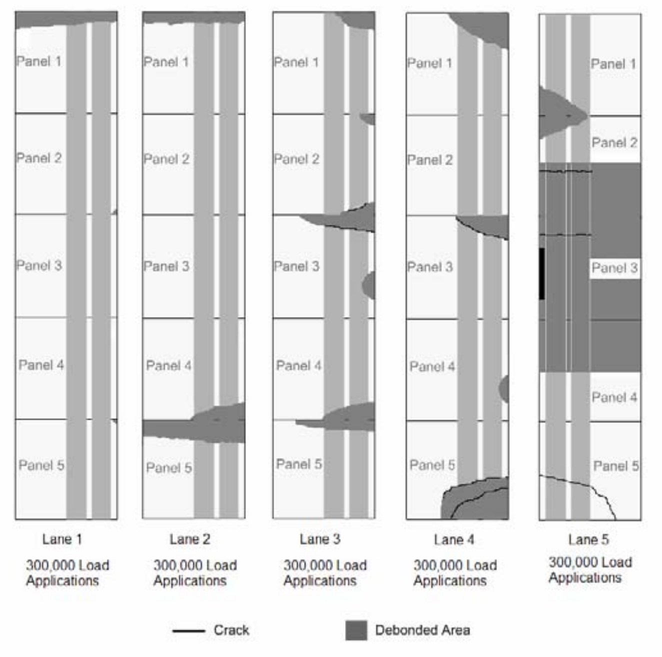
Lanes 1, 4, and 5 in test areas TA2 and TA3 were constructed to influence of variable bond conditions. Due to improper bond preparation in TA2, Lane 1 behaved as an unbonded or partially bonded section, Lane 4 was partially bonded and Lane 5 was unbonded. In TA3, Lanes 1 and 4 were fully bonded while Lane 5 included panels that were unbonded and partially bonded. These test areas are used in the subsequent discussion to evaluate the influence of bond conditions on the type and severity of distresses.

Figure 4 illustrates the final distresses in TA2 and TA3. In test area TA2, while there was a fair amount of overall distress (debonding and cracking) for Lane 4 (partially bonded) it was less than both Lane 1 and Lane 5. The distresses in Lane 4 were mainly debonding at the corners. Lane 1 (partially or unbonded) actually performed worse than Lane 5 (unbonded) which was deliberately debonded. However, this may have been due to the fact that there were significantly fewer load applications in Lane 5. Had the same number of load applications been applied to Lane 5 more distresses likely would have been noted.



(a)

FIGURE 4 Final Distress Comparisons for Test Areas 2 and 3 (a) TA2 (b) TA3



(b)

FIGURE 4 (continued) Final Distress Comparisons for Test Areas 2 and 3 (a) TA2 (b) TA3

For TA3, the distress summary presented in Figure 4 indicates that Lane 1 (fully bonded) had very little distress. The debonding noted was likely the result of edge affects of the APT test pit. Lane 4 (fully bonded), neglecting the distresses likely caused by the APT edge effects (in Panels 1 and 5), did have some debonding and one of the debonded areas did crack at the corner. In Lane 5, the degree of distress was greater than in Lane 1 and Lane 4. The UTW in Lane 5 cracked in the unbonded section. Additionally there was debonding noted in the partially bonded areas under Panels 1 and 2 in Lane 5. These results indicate that, as expected the interfacial bond conditions do influence the type and severity of distresses in the UTW layer.

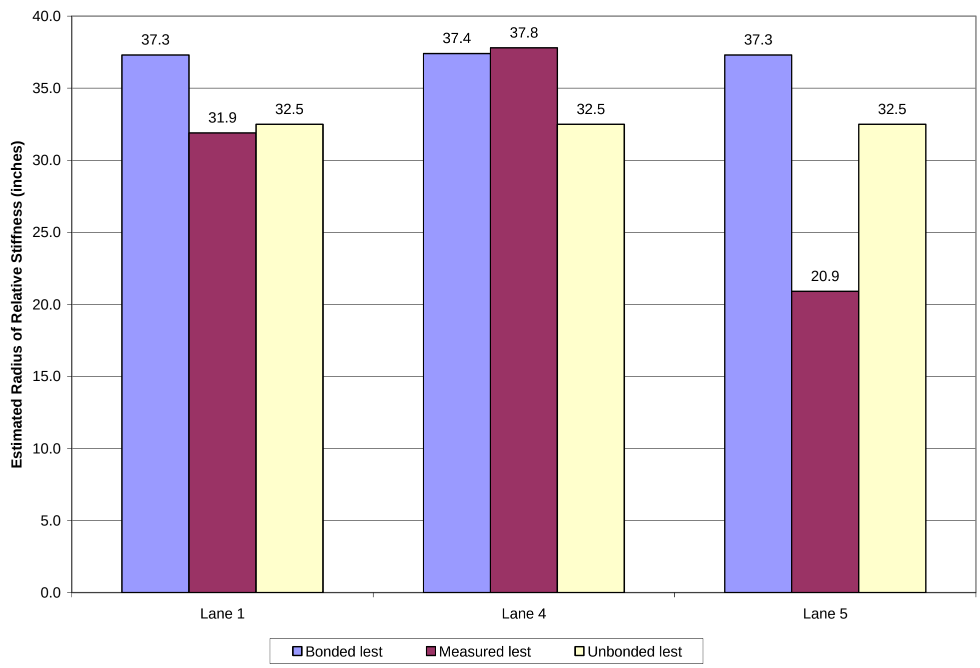
Influence of Interfacial Bond Conditions on Radius of Relative Stiffness

Li et al. [2004] reported that the degree of bonding of a UTW pavement could be determined using pavement deflection analysis. The methodology employed was very similar to the comparison that was just conducted in that the limits of the fully bonded and unbonded condition were used to frame the extremes expected for a given parameter. However, in Li's method the parameter utilized was the pavement deflection (as opposed to the radius of relative stiffness). Once the deflections were determined, a factor representing the degree of bonding was calculated. It may be possible to employ a similar methodology using the estimated radius of relative stiffness as the parameter. Additionally, the PCA design equations utilize the radius of relative stiffness to determine the pavement stresses and strains. It has been suggested by Newbolds [2007] that the estimated radius of relative stiffness determined by pavement deflection analysis can be integrated into the UTW design methodology.

The estimated radius of relative stiffness determined by pavement deflection analysis is dependent upon the pavement deflections, as noted previously. The relationship between the two is an inverse relationship in that higher deflections (from a more flexible pavement) result in a lower radius of relative stiffness. Layered pavement sections that act as a composite (those which have a good bond between the layers) have lower deflections and thus, have a higher radius of relative stiffness. This effect can be illustrated by using layer elastic analysis.

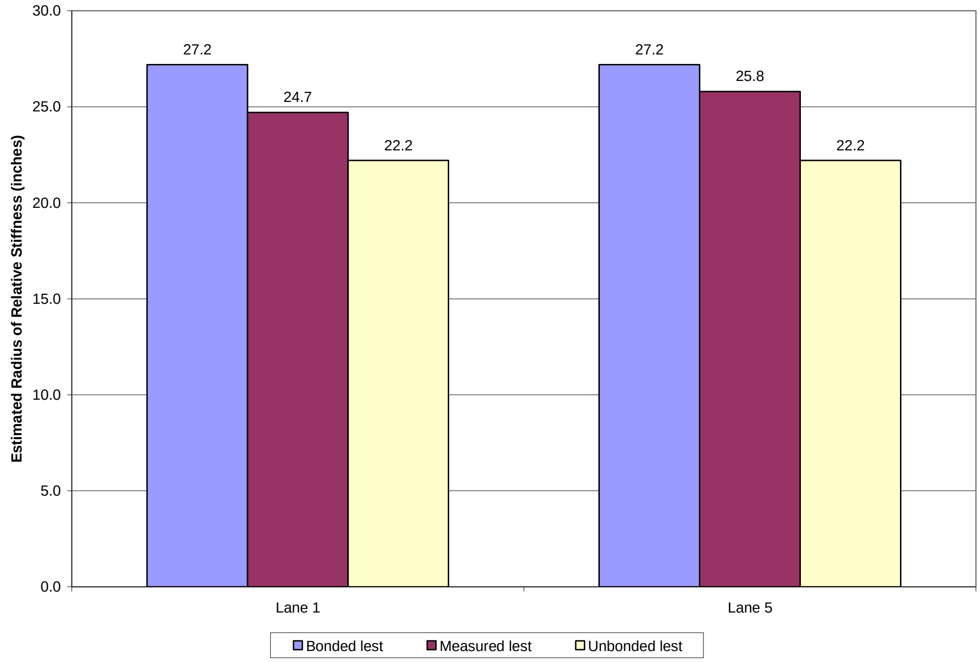
Using the WinJULEA program, the deflections for TA2 Lane 1 were calculated, using the methodology described above, first with a full bond, and then again without a bond. These deflections were input in the AASHTO equations to calculate the estimated radius of relative stiffness. These values are referred to as the 'theoretical'values. The value calculated for the fully bonded condition was 37.3 inches (947 mm) versus the unbonded condition which was 32.5 inches (825 mm).

The estimated radii of relative stiffness, l_{est}, were calculated using FWD data for TA2 Lanes 1 (effectively partially bonded), 4 (partially bonded), and 5 (unbonded) for fully bonded and unbonded conditions. The same values were also calculated for Lanes 1 (fully bonded) and 5 (FWD deflections would reflect partially bonded and unbonded conditions in TA3. These data are referred to as the 'measured' data. The theoretical and measured data were combined and are presented in Figure 5.



(a)

FIGURE 5 Comparisons of Measured Radii of Relative Stiffness (lest) with Theoretical Bonded and Unbonded Radii of Relative Stiffness (a) TA2 (b) TA3



(b)

FIGURE 5 (continued) Comparisons of Measured Radii of Relative Stiffness (lest) with Theoretical Bonded and Unbonded Radii of Relative Stiffness (a) TA2 (b) TA3

As shown in this figure, for TA2 the data indicate that the radius was lower than the fully bonded condition for Lane 1. Again, this would be reflective of the weak bond conditions actually observed in this test lane. For TA2 Lane 4, the measured results indicate a fairly high value for the radius. This does not follow what would be expected for a partially bonded lane and may indicate that this method is not sensitive to partially bonded conditions. However, the radius for TA2 Lane 1 is less than the radius for TA2 Lane 4. (Actual Iowa shear strength values from cored specimens also reflected this trend.) The results for TA2 Lane 5 show a very low value of the radius for the unbonded lane. While this would be expected, the value is still significantly less than the lowest theoretical value. This may be the result of the dynamic loading of the pavement in the FWD test. The dynamic loading imparted by the FWD may cause greater deflections than those expected with a theoretical static load. The results further indicate that using these data to characterize the degree of bonding would not be possible since the actual values of the radii are outside the theoretical range for all lanes.

For Lane 1 in TA3, the value of the radius for the fully bonded condition falls within the expected range of values. However, the value was expected to be closer to the theoretical value for the fully bonded condition. The value of the radius in TA3 Lane 5 was also between the two expected values. Because Lane 5 reflected partially bonded and unbonded conditions, it was expected that the value of the radius would be less than the value of the radius for the fully bonded condition in Lane 1. However, the bond shear strengths for cores taken from these lanes were also very close. Thus, the result that the measured radii were close on TA3 Lanes 1 and 5 was corroborated by the actual bond strengths. In regards to the degree of debonding, given the position of these results within the expected range, approximately half way, it does not appear that this methodology would provide a good estimate of the degree of bonding, which in this case would be 50 percent.

CONCLUSIONS

The interfacial bond conditions did affect the strain distribution and maximum load-induced strains. The measured strains in the bonded lanes matched the theoretical values well. There was some discrepancy between the measured and theoretical unbonded strain distributions. Further analysis into this discrepancy indicated that even in an unbonded condition there is moderate level of friction between the UTW and HMA layers. This is likely due to the increase in friction between the two layers as a result of the traffic load.

The amount of distress generally followed the expectations based on the interfacial bond condition. Those lanes having a lower degree of bonding (partially bonded or unbonded conditions) typically exhibited higher amounts of distress. Conversely, the lanes with the highest degree of bonding (fully bonded) typically showed the lowest degree of distress.

The interfacial bond conditions did have an influence on the estimated radius of relative stiffness. The results indicate that the radius can be used as a method to determine whether or not the UTW was bonded. However, using the radius to determine the degree of bonding did not provide very good results. The relative values of the radius matched the trend exhibited by the bond strengths for cores taken from the same lanes.

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