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Performance Analysis of Ultrathin Whitetopping Intersections on US-169

Elk River, Minnesota

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The Minnesota Department of Transportation constructed an ultrathin whitetopping (UTW) project at three consecutive intersections on US-169 at Elk River, Minnesota, to gain more experience with both the design and the performance of UTW. Distinct cracking patterns developed within each test section. The UTW test sections with a 1.2- × 1.2-m (4- × 4-ft) joint pattern included corner breaks and transverse cracks. Corner breaks were the primary distress in the test section with a 1.8- × 1.8-m (6- × 6-ft) joint pattern, although very little cracking was exhibited. The Minnesota Road Research Facility UTW test sections on I-94 allow comparisons of the same UTW design on hot-mix asphalt (HMA) pavements with different structural capacities to be made. The strain and deflection measurements emphasize the importance of the support provided by the HMA layer. A reduction in this support occurs when the temperature of the HMA is increased or when the HMA begins to ravel. During evaluations of whether UTW is a viable rehabilitation alternative, cores should be pulled from the pavement to determine if the asphalt is stripping and if the asphalt layer has adequate thickness. UTW can be successfully placed on as little as 76 mm (3 in.) of asphalt, if the quality of the asphalt is good. The cores should also reveal whether the asphalt layer is of uniform thickness and whether stripping and raveling have occurred. If the asphalt layer is of uniform thickness and stripping and raveling have not occurred, UTW is a good option for use in the rehabilitation of asphalt pavements.

Whitetopping refers to the placement of a thin concrete overlay directly on top of an existing distressed hot-mix asphalt (HMA) pavement. A concrete overlay ranging between 50 and 100 mm (2 and 4 in.) thick is commonly referred to as ultrathin whitetopping (UTW). For long-term performance, the overlay must bond to the underlying asphalt so that the two layers respond in a monolithic manner, thereby reducing load-related stress. A short joint spacing is also used to help reduce curling and warping as well as bending stresses. Typical applications would include low- to medium-volume pavements where rutting, washboarding, or shoving is present, such as intersections, bus stops, airport aprons, taxiways, and parking lots (*1*).

The Minnesota Department of Transportation constructed UTW test sections at three consecutive intersections on US-169 in Elk River, Minnesota, to gain more experience with both the design and performance of UTW. The test sections were located on the outer southbound lane of US-169 in Elk River at the intersections of Jackson, School, and Main Streets (Figure 1). All three intersections have traffic signals. The speed limit on US-169 changes from 89 to 72 km/h (55 to 45 mph) just north of Jackson Street as the traffic approaches the city of Elk River. Many of the commercial trucks traveling southbound on US-169 are coming from the gravel pits, concrete plants, and waste disposal facilities just north of this intersection.

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PREEXISTING PAVEMENT STRUCTURE

The original pavement was constructed in 1961 on a sandy subgrade and consisted of a 100-mm (4-in.) HMA surface on 280 mm (11 in.) of densely graded aggregate base. In 1991, the pavement was overlaid with HMA. The HMA layer was 152 mm (6 in.) before placement of the UTW.

A distress survey was performed on each 244-m (800-ft) section before construction of the overlay. Transverse joints were sawed into the HMA pavement approximately every 9 m (30 ft). The average transverse crack and joint spacings at the Jackson Street, School Street, and Main Street test sections were 5 m (17 ft), 6 m (20-ft), and 7 m (22-ft), respectively. The cracks were all low to medium in severity. The HMA was raveled in areas, especially along the outer edge. Severe rutting [greater than 32 mm (1.25 in.)] and shoving were also present before milling, as a result of the stopping and starting of heavy trucks at each intersection (Figure 2).

Falling weight deflectometer (FWD) testing was performed in the wheelpath for each test section at 15-m (50-ft) intervals on September 4, 1997, just before placement of the concrete overlay. The deflection measured directly under the load plate provides an indication of the stiffness of the pavement structure. The averages of three normalized deflections measured for a 40-kN (9-kip) load for each of the test sections are provided in Figure 3*a*. All measured deflections were adjusted to the deflections expected at a middepth asphalt temperature of 20°C (68°F). These adjustments were made by using the procedure presented by Lukanen et al. (*2*). The middepth asphalt temperature at the time of testing was estimated by using BELLS3 (*2*). The deflections are similar for all test sections but are slightly lower at the north end of the Jackson Street test section, indicating that the pavement structure might be slightly stiffer in this area.

The AREA basin factor was calculated for the deflection data (*3*). AREA is derived from the area of the deflection basin curve normalized with respect to the deflection recorded directly under the load plate and represents the ratio of the stiffness of the pavement to the stiffness of the subgrade. The AREA factor was calculated by using the average for the three deflections normalized to a 40-kN (9-kip) load. The middepth asphalt temperature during testing was estimated by using BELLS3 (*2*). The AREA basin factors were adjusted to a middepth asphalt reference temperature of 20°C (68°F) by the procedure presented by Lukanen et al. (*2*). The AREA basin factors calculated for each test section are provided in Figure 3*b*. The ratio between

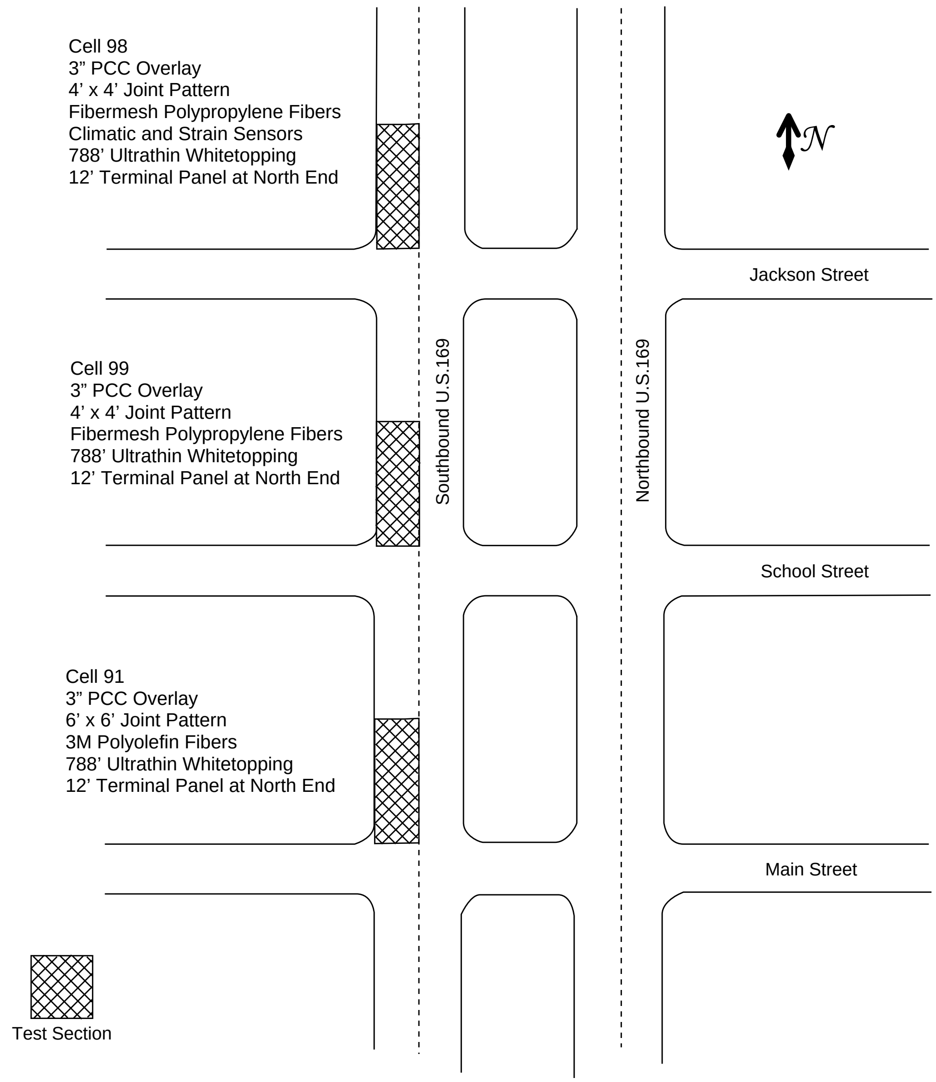


FIGURE 1 US-169 project layout. Cell numbers are Minnesota Road Research designations. (PCC = portland cement concrete.)

the stiffness of the pavement and the stiffness of the subgrade appears to be relatively constant within and between each test section. However, AREA basin factors calculated at the north end of the Jackson Street test section are slightly higher, indicating that the increase in the stiffness of the pavement structure in this region, as reflected in Figure 3*a,* might be attributed to a stiffer pavement surface.

An attempt was made to backcalculate the resilient modulus of each layer by assuming that each layer consisted of a linear elastic homogeneous material. It was readily apparent that the responses of the various layers were nonlinear. The nonlinearity in the upper layers was in the form of stress stiffening, and the nonlinearity in the lower layers was possibly in the form of stress softening. The analysis did indicate that the subgrade was very stiff, as would be expected, because the subgrade consisted of a sandy gravel. The base was constructed of densely graded aggregate containing a large amount of fine material, so the stiffness of this material is most likely not as high as desired. The deflection data also indicated the presence of stripping in locations. This was verified when cores were taken from the test sections.

DESCRIPTION OF TEST SECTIONS

The UTW sections were constructed only in the outside lane in the southbound direction. The first 240 m (788 ft) north of each intersection was overlaid with 75 mm (3 in.) of fiber-reinforced concrete. The concrete used for the Jackson and Main Street UTW sections contained polypropylene fibers, and that used for the School Street intersection contained polyolefin fibers. The Jackson and Main Street test sections had 1.2- × 1.2-m (4- × 4-ft) panels. The School Street intersection had 1.8- × 1.8-m (6- × 6-ft) panels. The 3.7 m (12 ft) on the north end of each test section was milled to a depth of 203 mm (8 in.) (Figure 1). The purpose of the thicker section was to reduce the damage that would occur as heavy trucks come off the HMA pavement onto the UTW. Sensors for temperature (type-T thermocouples), dynamic strain (Tokyo Sokki PML-60), and static strain (Geokon VCE 4200 vibrating wire strain gauges) were installed approximately 37 m (120 ft) north of the Jackson Street intersection. Additional information on the construction of these test sections can be obtained elsewhere (*4*).

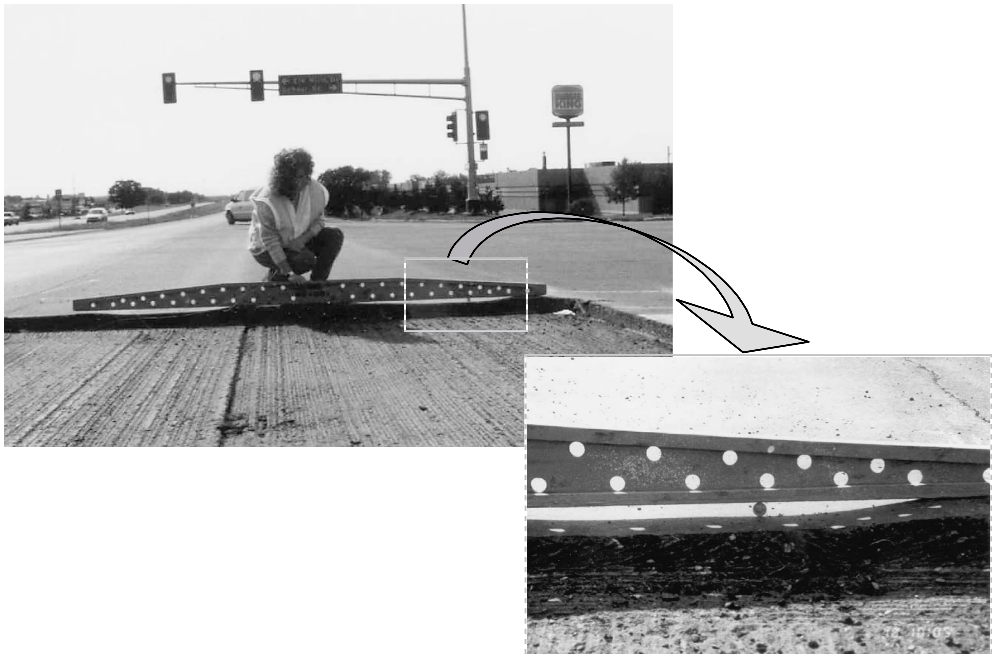


FIGURE 2 Ruts at the Jackson Street intersection before placement of the UTW overlay.

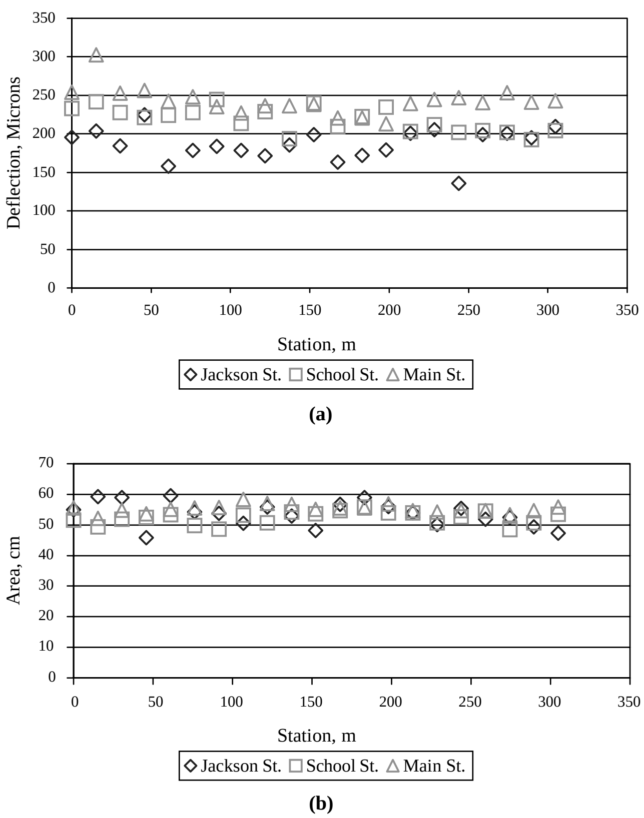


FIGURE 3 From FWD testing: (*a*) FWD deflections under load plate measured for the HMA pavement; (*b*) AREA basin factors calculated for deflection data for each test section before placement of the overlay. The values are averages normalized to 40-kN (9-kip) loads and are corrected for temperature.

CONSTRUCTION

On September 17, 1997, all three intersections were milled to maintain existing elevations, swept twice, and air blasted. The milling enhanced the bond by providing a macrotexture from the ridges milled into the surface and the freshly fractured aggregate surfaces that were exposed. After the intersections were milled, the underlying HMA at the Jackson Street intersection appeared to be more severely raveled than the HMA at the other two intersections, especially along the longitudinal seams between the roadway and the shoulder. Some of the areas were so severely raveled that air blasting removed pieces of HMA from the pavement surface. No preoverlay repairs were performed. The concrete was placed directly on the milled surface without a tack coat or whitewash.

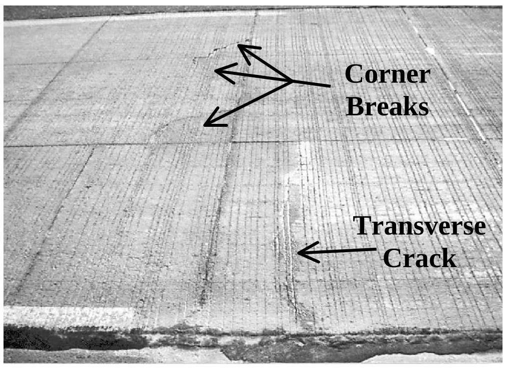
TRAFFIC

The test sections on US-169 were in service between September 1997 and September 1999. The one-way average annual daily traffic (AADT) was 16,000 in 1997 for this section of roadway, with 8% being trucks. The AADT grew to 17,000 by 1999. Forty-nine percent of these trucks are categorized as five-axle semitrailers.

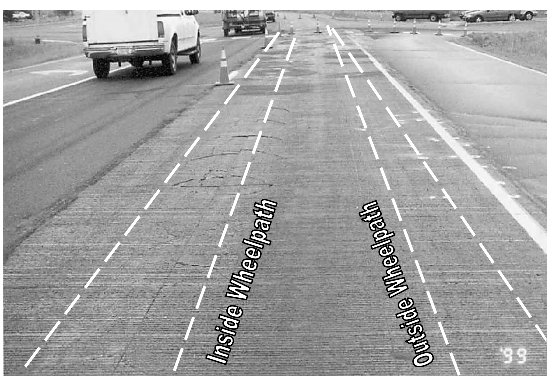
PERFORMANCE

Increasing the concrete thickness of the first 3.7 m (12 ft) of each test section to 203 mm (8-in.) successfully prevented any distress from occurring on the test sections as the vehicles came off from the HMA pavement onto the UTW. The most heavily distressed area in each of the test sections was just before the intersection. The change in vehicle speed is the greatest in this location because of vehicle acceleration and deceleration when the traffic light changes.

The cracks observed in the UTW test sections with the 1.2- × 1.2-m (4- × 4-ft) joint pattern included corner breaks and transverse cracks. The corner breaks occurred primarily along the inside longitudinal joint and the lane–shoulder (L/S) longitudinal joint. Many of the corner breaks that developed along the inside longitudinal joint did not appear until 1999. The inside longitudinal joint lies directly in the inside wheelpath, resulting in high edge and corner stresses. Transverse cracks developed in the panels adjacent to the shoulder. The transverse cracks typically develop 0.4 m (1.3 ft) away from the transverse joint, which is approximately one-third of the length of the panel. These crack patterns are shown in Figure 4*a* and 4*b*.



(a)



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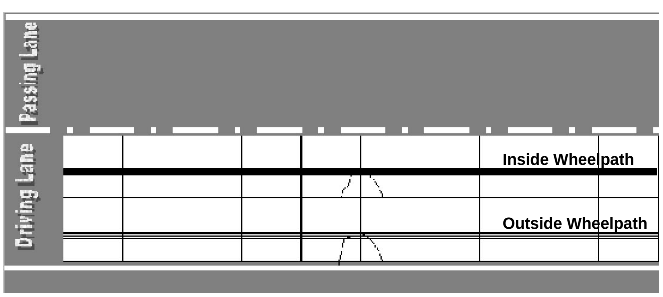

(c)

FIGURE 4 Crack patterns: (*a*) transverse crack and corner breaks in the Jackson Street test section (March 30, 1998); (*b*) corner breaks in the inside wheelpath at the Jackson Street test section (July 20, 1999); and (*c*) typical distress patterns that developed in the Main Street test section.

The Main Street test section was constructed by using a 1.8- × 1.8-m (6- × 6-ft) joint pattern. Corner breaks were the primary distress that developed in this test section, although very little cracking was exhibited. The corner breaks were typically located in the outside panel adjacent to the L/S joint and intersected the transverse joint in the wheelpath. Corner breaks in the inside panel typically intersected the transverse joint in the wheelpath but then intersected the longitudinal joint separating the two panels. The corner breaks exhibited in both the inside and the outside panels intersected the longitudinal joint nearest each wheelpath (Figure 4*c*). The Main Street test section performed significantly better than the Jackson and School Street intersections because the longitudinal joint did not lie in the inside wheelpath for a 1.8- × 1.8-m (6- × 6-ft) joint pattern. This significantly reduced the edge and corner stresses.

UTW AT MINNESOTA ROAD RESEARCH FACILITY ON I-94

In October 1997, approximately the same time that the US-169 test sections were constructed, six other whitetopping sections were constructed on I-94 at the Minnesota Road Research Facility (MnROAD). I-94 is a heavily trafficked road with average daily traffic of approximately 25,000, of which 12% to 13% is truck traffic. An Interstate highway is not a typical application for UTW, but this location offered the opportunity to perform an accelerated test with UTW because design loads comparable to those found at a more traditional UTW site could be accumulated more rapidly. The HMA pavement was in relatively good condition before placement of the overlay. Low-severity transverse cracks had developed every 4.8 m (15 ft), and approximately 6 mm (0.25 in.) of rutting had developed in the right wheelpath of the driving lane. The test sections at MnROAD allow comparisons of the same UTW design on HMA pavements with different structural capacities to be made. Comparisons were made between three of the test sections included in the MnROAD whitetopping study and the US-169 test sections. A summary of the design features of these sections is as follows: Cell 93, 102-mm × 1.2-m × 1.2-m (4-in. × 4-ft × 4-ft) panels and concrete with polypropylene fibers on 241 mm (9.5 in.) of asphalt concrete (AC); Cell 94, 75-mm × 1.2-m × 1.2-m (3-in. × 4-ft × 4-ft) panels and concrete with polypropylene fibers on 267 mm (10.5 in.) of AC; and Cell 95, 75-mm × 1.5-m × 1.8-m (3-in. × 5-ft × 6-ft) panels and concrete with polyolefin fibers on 267 mm (10.5 in.) of AC. Additional information on the construction and performance of these test sections is provided elsewhere (*4, 5*).

Comparison of the findings from the preoverlay distress survey with those from the distress surveys performed after the overlay was constructed revealed that none of the transverse joints or cracks in the HMA were reflected into the overlay for any of the test sections. Reflective cracks did develop in the 76- and 102-mm (3- and 4-in.) UTWs constructed on I-94. The same joint patterns used on US-169 were also used for construction on I-94. The difference in performance can be attributed to the fact that the UTW on US-169 was placed on 76 mm (3 in.) of HMA that exhibited signs of raveling and the fact that the UTW on I-94 was constructed on 254 mm (10 in.) or more of quality HMA. This resulted in a higher bond strength and a greater structural rigidity in the HMA layer, producing higher tensile stresses at the bottom of the UTW in the regions of the cracks in the HMA. The potential for reflective cracking appears to develop when the flexural stiffness of the HMA pavement approaches that of the overlay.

FWD DATA

FWD testing was performed at various times of the year in an attempt to capture the seasonal effects on the relationship between the applied load and the resulting deflection. Graphical depictions of the FWD test locations for each test section are provided in Figure 5.

A discontinuity typically appeared in the deflection basin at the location of the transverse joint if the applied load was within 600 mm (24 in.) of the joint at the Jackson and School Street intersections. An exception to this occurred when FWD testing was first performed in March and April 1998, approximately 5 months after the test sections were constructed. The smooth deflection basins obtained during this time indicate that most joints did not crack before March 1998. The deflection data collected during July 1998 indicate that most of the joints did crack sometime between March and July 1998. The joint spacing for the Main Street test section was too large for the transverse joint to be within 600 mm (24 in.) of the applied joint except when testing was preformed adjacent to the transverse joint. Therefore, it was not possible to estimate when the joints cracked on the basis of the FWD data for the Main Street test section.

Graphs of the average normalized deflections measured directly under the load plate for a 40-kN (9-kip) FWD load are provided in Figures 6 to 10. The lowest deflections were measured in the winter, when the subgrade was frozen and the asphalt was stiff. Deflections in the same locations at other times of the year were as much as six times higher. The highest deflections were typically measured in the summer, when the asphalt was less stiff, as would be expected. This trend was more prevalent at the Jackson and School Street intersections and when loads were applied at the edge or in the cor-

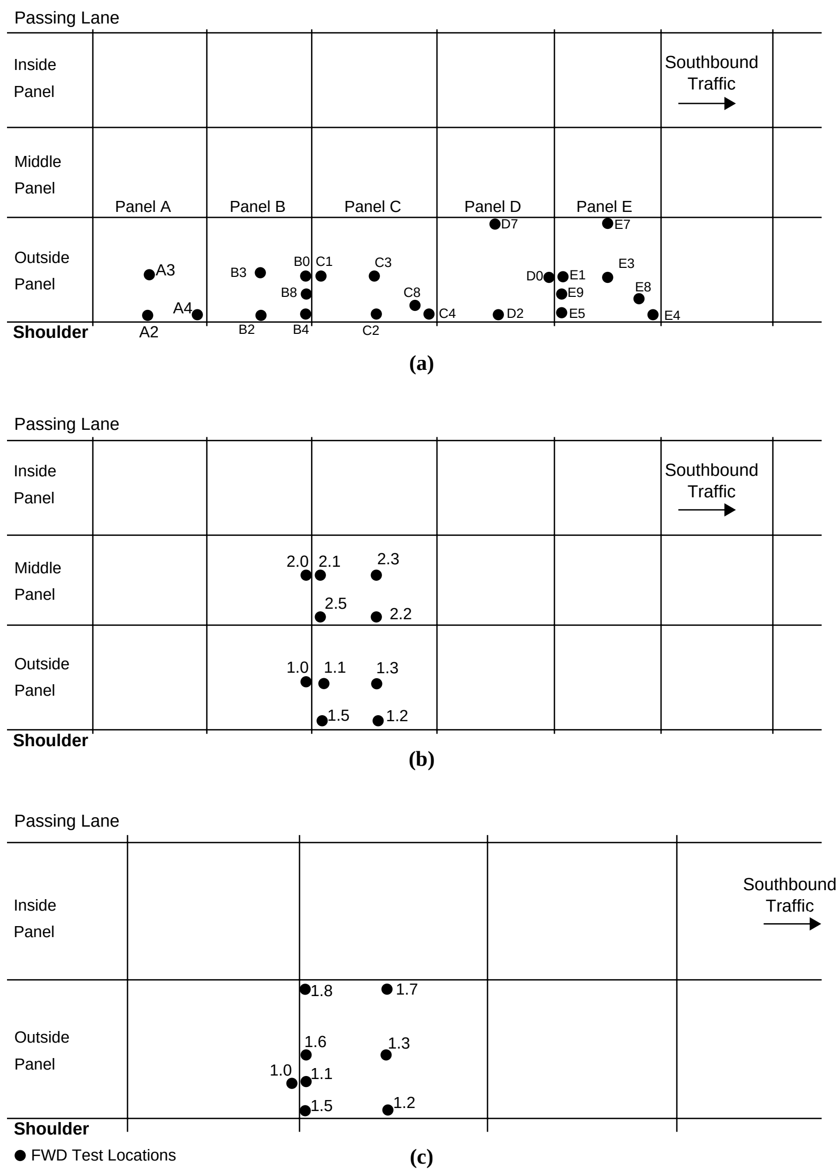

FIGURE 5 US-169 UTW FWD test locations at the *(a)* Jackson Street intersection with 3-in.-thick UTW with 4- × 4-ft panels; *(b)* School Street intersection with 3-in.-thick UTW with 4- × 4-ft panels; and *(c)* Main Street intersection with 3-in.-thick UTW with 6- × 6-ft panels.

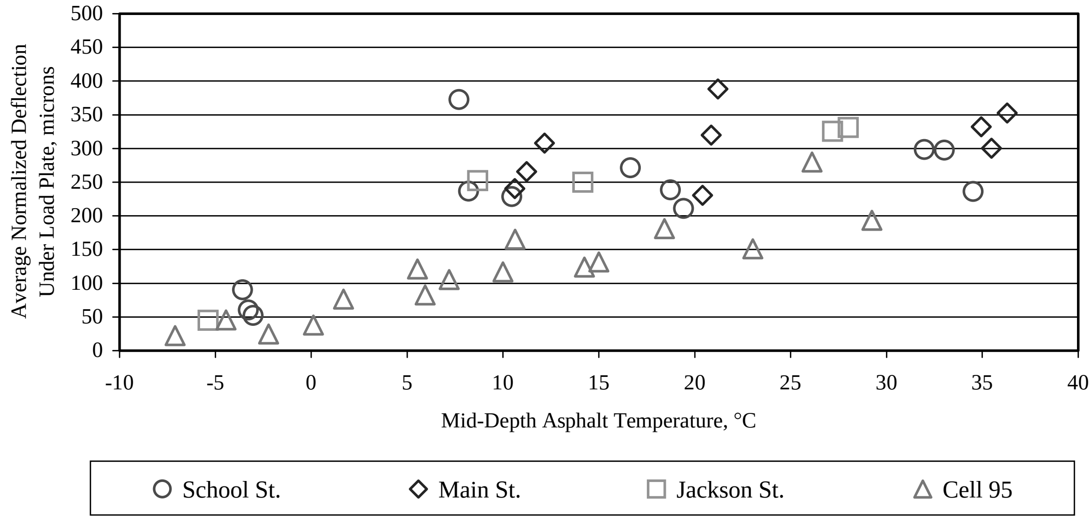

FIGURE 6 Average normalized deflections measured directly under the load plate for a 40-kN (9-kip) FWD load in the corner.

ner, where the response of the slab is more heavily influenced by changes in support conditions.

The lowest deflections were measured at midpanel, where the distribution of the load is not obstructed by a discontinuity. The highest deflections were measured in the corner and along the L/S joint. The areas with the higher deflections also corresponded to the areas within the panel where distresses developed: corner breaks and midpanel cracks that most likely initiated at the L/S joint. These locations also exhibited the largest amount of scatter between the deflections measured at the same time and same location but for different panels. This indicates that the support conditions vary more in the vicinities near joints. The joints allow water to enter the pavement structure, which can then lead to raveling of the asphalt at the concrete–asphalt interface and nonuniform bond conditions. The cores taken from the test sections also indicated that the HMA ravels at a faster rate along the joints. The variability within each test location and test section also increased with increasing asphalt temperatures.

The deflections were significantly lower on the MnROAD test sections than on the US-169 test sections because the existing asphalt was thicker and less deteriorated before placement of the overlay. The condition of the asphalt on the MnROAD test sections was uniformly good throughout the project, unlike at Elk River. This resulted in more consistent deflection measurements within each MnROAD test cell for each test location. The magnitude of the deflection was predominately a function of the thickness of the asphalt and not the overlay thickness or joint spacing for the UTW designs included in this study. This emphasizes the need to ensure that the asphalt layer is sufficiently thick before UTW is considered as a rehabilitation alternative. The MnROAD test sections not only had lower deflections, but the relationship between deflection and temperature was

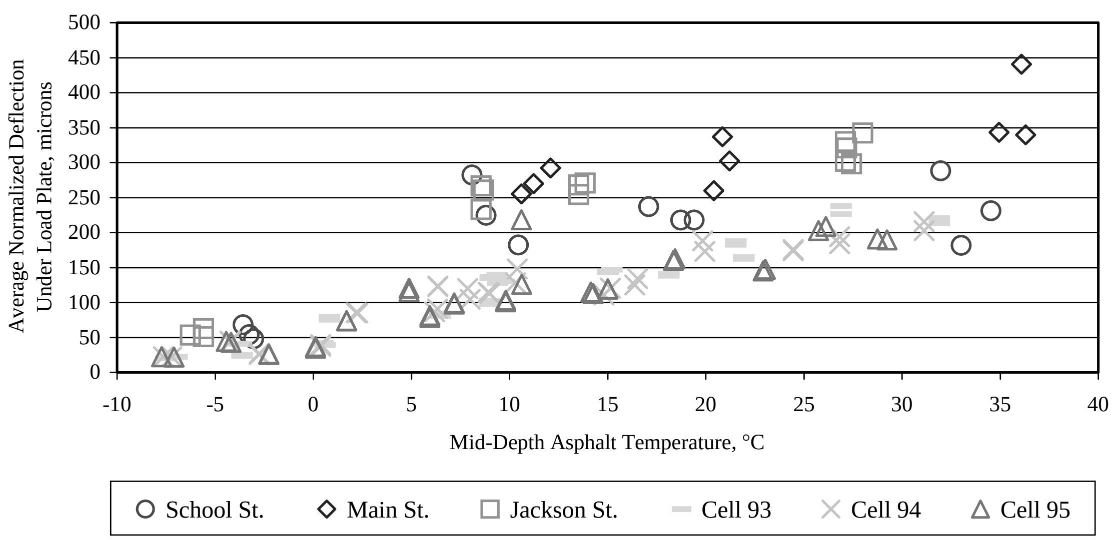

FIGURE 7 Average normalized deflections measured directly under the load plate for a 40-kN (9-kip) FWD load along the L/S edge.

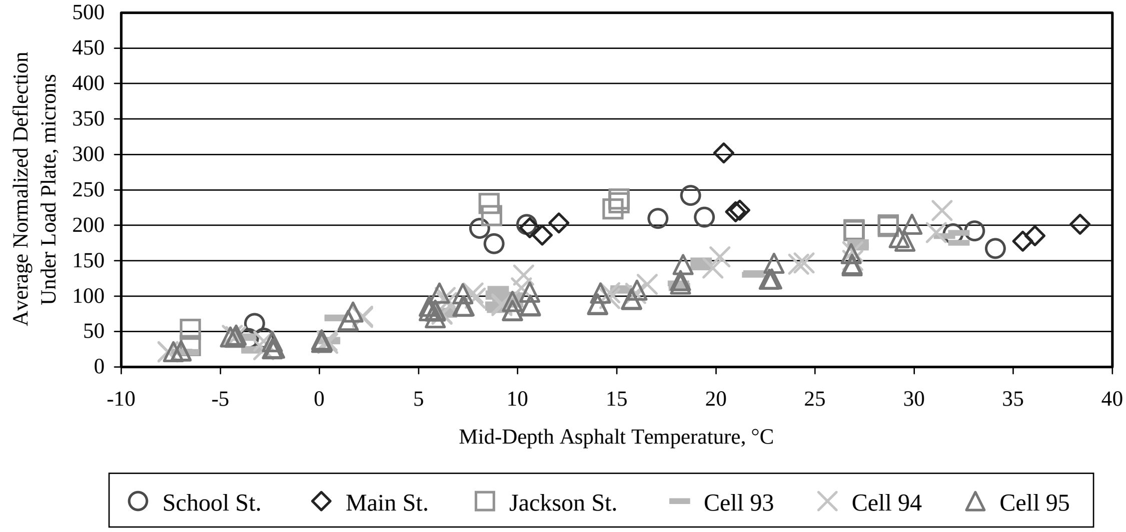

FIGURE 8 Average normalized deflections measured directly under the load plate for a 40-kN (9-kip) FWD load at midpanel.

also relatively linear compared with the relationship for the deflections measured at US-169. For the test sections on US-169, the rate at which deflections increased with increasing temperature decreased as the temperature of the HMA increased until a peak deflection was eventually reached. The MnROAD test sections are still in place after the accumulation of 4 million equivalent single-axle loads, showing that the lower deflections will result in an extended pavement life.

MEASURED DYNAMIC STRAINS

The Jackson Street test section was instrumented with dynamic strain gauges. Strain gauges were located at the bottom of the UTW and approximately 25 mm (1 in.) from the surface of the overlay. At each location all sensors except the static strain sensors were replicated. Dynamic strains were measured in conjunction with FWD testing. The average of three strain measurements resulting from the application of a 40-kN (9-kip) load directly over each sensor was plotted against the temperature measured at the middepth of the HMA layer. The results are provided in Figure 11. Positive values represent tensile strains, and negative values are compressive strains.

The largest strain measured was along the L/S joint, where the edge has less support than that available along the interior edges. Very few data were obtained at the bottom of the UTW at this location because the strain gauges failed early. See Figure 5 for the locations of Panels B and C. The strain measured in Panel C was consistently higher than that measured in Panel B. Coring of each location revealed that the HMA was severely stripped. The HMA from the core in Panel B

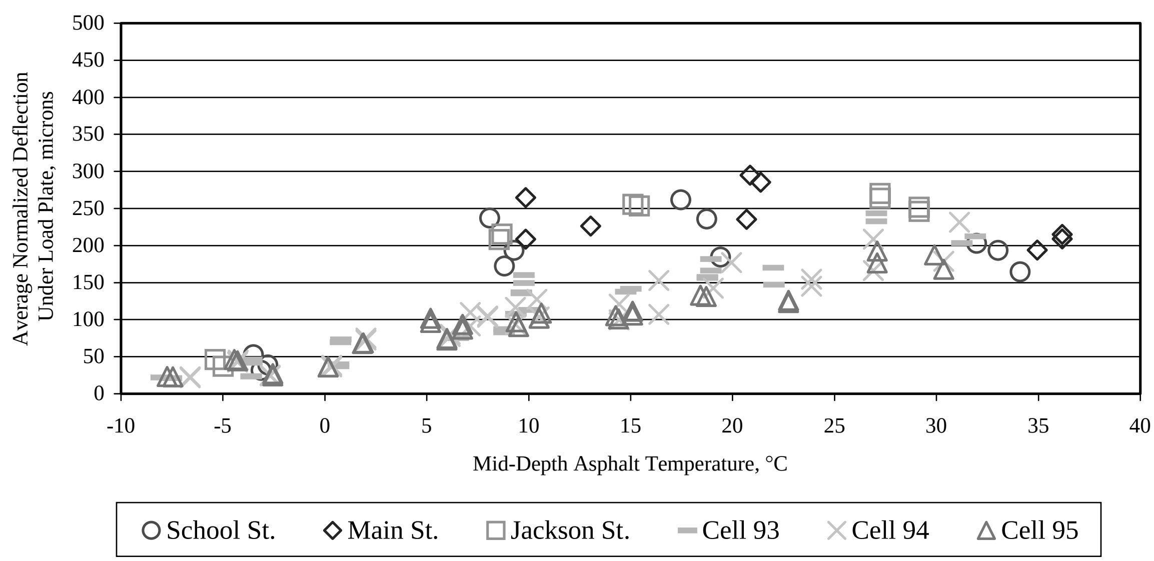

FIGURE 9 Average normalized deflections measured directly under the load plate for a 40-kN (9-kip) FWD load in the wheelpath.

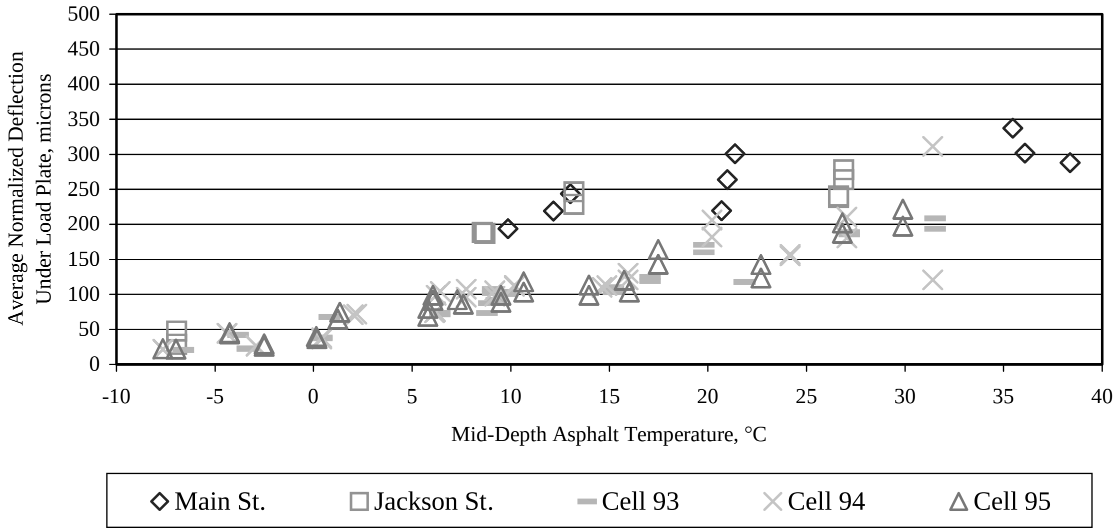

FIGURE 10 Average normalized deflections measured directly under the load plate for a 40-kN (9-kip) FWD load along the inside longitudinal joint.

had turned completely into unbound aggregate, and approximately 75% of the HMA from the core in Panel C was stripped to the point of being unbound aggregate. The combination of edge loading and the fact that the HMA below the overlay was severely stripped and raveled resulted in high strains along the L/S joint.

The strains measured at the corner in Panel E are higher than those measured at the corner in Panel C. The higher strains can be explained by the fact that the HMA at the corner in Panel E is more severely raveled than the HMA at the corner in Panel C, resulting in a loss of support and a reduction in the bond between the two layers. The strain gauges placed in the corner were actually located 305 mm (12 in.) away from the transverse and longitudinal edges and not directly in the corner. The HMA there was in significantly better condition than the HMA along the L/S joint. A core was pulled directly from the

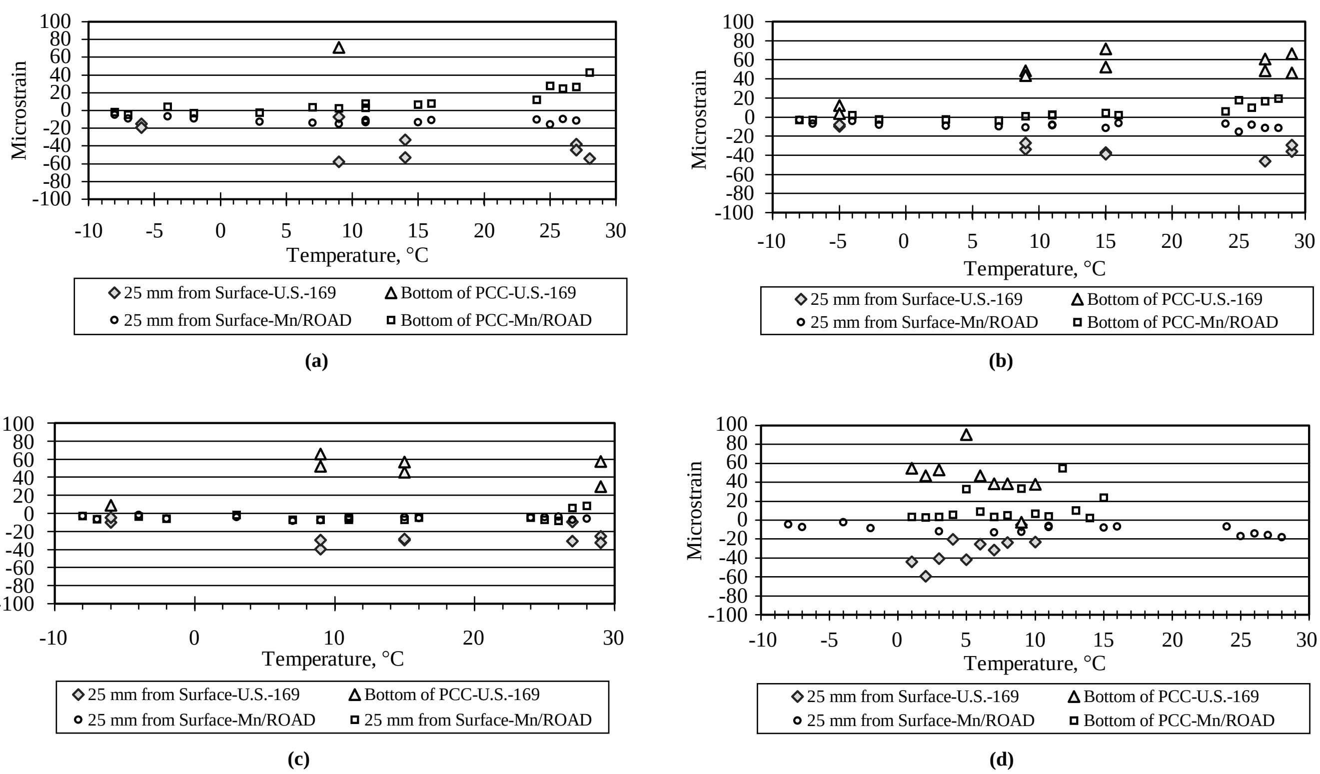

FIGURE 11 Comparison of strains measured on I-94 and US-169 from application of a 40-kN (9-kip) load: (*a*) L/S longitudinal joint; (*b*) loaded side of the transverse joint and in the wheelpath; (*c*) corner; and (*d*) midpanel.

corner of Panel E, and the asphalt from the HMA was completely stripped and raveled, showing the progressive deterioration in the asphalt when the pavement approached the joints.

The strain measured adjacent to the transverse joint in the wheelpath in Panel D was consistently higher on the top sensor than that measured in Panel C because the sensor in Panel D was approximately 10 mm (0.4 in.) closer to the pavement surface. Approximately 30% of the strain is transferred across the joint between Panels B and C, and approximately 40% of the strain is transferred across the joint between Panels D and E. Cores taken from both locations revealed that the UTW was bonded to the HMA.

The strains measured at midpanel typically ranged between 35 and 60 microstrains at the top of the UTW when the HMA temperature was approximately 10°C (50°F) or greater, and the strains measured at the bottom of the overlay ranged between 20 and 45 microstrains. The strains measured at midpanel in Panel C were significantly higher than those measured at midpanel in Panel E. Cores were pulled from the midpanels of both Panels E and C, and the bond strength of the core from Panel E was measured by the Iowa direct shear test (Iowa Test Method 406-C). The shear strength measured for the core pulled from Panel E was 270 psi. A weaker bond was found in the core taken from Panel C. The HMA separated from the UTW before the bond strength could be measured.

The strain measurements emphasize the importance of the support provided by the HMA layer. A reduction in this support occurs when the temperature of the HMA is increased or when the HMA begins to ravel. Raveling might also result in a reduction of the bond strength between the two layers but typically initiated at the bottom of the HMA layer. The results from the strain measurements and for the cores pulled from the test section indicate that the HMA ravels at a faster rate along the joints, where there is greater access for water to enter the pavement structure compared with that at the center of the panel. The L/S joint is the most difficult to keep sealed, and therefore, the HMA along this joint is more susceptible to stripping and raveling. Consideration should be given to sealing the joints to limit water from coming into contact with the HMA layer. The short joint spacings typically associated with UTW result in small joint movements, making it easier to seal joints as narrow as 3 mm (0.125 in.). Also, during the original construction of the HMA pavement it is more difficult to achieve the desired compaction along the longitudinal joint. Insufficient compaction results in a lower-quality HMA, resulting in reduced support and higher strains in the UTW overlay. The quality of the HMA along the longitudinal edge should be evaluated before it is determined whether an UTW is an appropriate rehabilitation alternative.

Dynamic strain was also measured in conjunction with FWD testing at various times of the year for the I-94 test section. The results are presented in Figure 11, along with the strains measured on US-169. The dynamic strains measured on I-94 were significantly lower than those measured on US-169 at all locations within each panel. The reduction in strain is a result of the increase in the thickness and quality of the HMA on I-94 and the increase in the bond strength between the two layers. These factors resulted in a shift of the neutral axis down into the HMA layers at HMA temperatures below 5°C (41°F), resulting in the generation of compressive strains at the bottom of the UTW when it was loaded. Increases in the temperature of the HMA also resulted in much smaller increases in strain on I-94 than on US-169. Significant increases in strain induced by the 40-kN (9,000-lb) FWD load began to occur at HMA temperatures below 10°C (50°F) on US-169, while significant increases in strain on I-94 were not seen until the temperature reached 25°C (77°F).

The strains measured on the I-94 test sections were consistently lower than those measured on the US-169 test sections, even when the measurements were made at higher HMA temperatures. The exceptions to this are the strains at midpanel. The strains measured at midpanel on I-94 approached those measured at midpanel on US-169. The UTW on I-94 is well bonded to the HMA, similar to the bond conditions found when cores were obtained at midpanel on US-169. In the regions along the edge of the panel where the UTW on US-169 was found to be unbonded, the strains on US-169 were significantly higher than those on I-94. This indicates that application of a load when the HMA temperature is high will produce similar strains in the UTW, regardless of the thickness of the HMA layer when a good bond is achieved. A more in-depth analysis of the deflections and strains measured on US-169 and I-94 can be found elsewhere (*5, 6*).

CONCLUSIONS

The construction of the UTW test sections on US-169 provided valuable insight into the construction and performance of UTW. Before the application of the overlay the HMA was severely rutted, with low- to medium-severity transverse cracks approximately every 6 m (20 ft). Raveling was also occurring, especially along the longitudinal seams. In the future UTW overlays should not be used on asphalt pavements with deteriorated longitudinal seams, because a good bond between the UTW and asphalt cannot be achieved. The high strains measured along the L/S joint on US-169 are indicative of the loss of bond between the two layers. UTW overlays should also not be used on pavements constructed of HMA that is susceptible to stripping.

Distinct cracking patterns developed within each test section. The UTW test sections with a 1.2- × 1.2-m (4- × 4-ft) joint pattern had corner breaks and transverse cracks. The corner breaks occurred primarily along the inside longitudinal joint and the L/S longitudinal joint, while the transverse cracks developed in the panels adjacent to the shoulder. The transverse cracks typically developed approximately one-third of the length of the panel away from the transverse joint. The Main Street test section with the 1.8- × 1.8-m (6- × 6-ft) joint pattern performed significantly better than the Jackson and School Street intersections because the longitudinal joint did not lie in the inside wheelpath. This significantly reduces the edge and corner stresses. Corner breaks were the primary distress that developed in the Main Street test section. Reflective cracking was not observed in any of the test sections, although reflective cracking has been found to occur in UTWs placed on thicker HMA pavements, such as on I-94.

The strains measured on I-94 were consistently lower than those measured on US-169, even when measurements were made at higher HMA temperatures. The reduction in strain was a result of the increase in the thickness and the quality of the HMA on I-94 and the increase in the bond strength between the two layers. Increases in the temperature of the HMA also produced much smaller increases in strain on I-94 than on US-169, except for the strains measured at midpanel. The strains measured at midpanel on I-94 approached those measured at midpanel on US-169. It was found that application of a load when the HMA temperature was high produced similar strains in the UTW, regardless of the thickness of the HMA layer when a good bond was obtained.

The strain measurements emphasize the importance of the support provided by the HMA layer. A reduction in this support occurs when the temperature of the HMA is increased or when the HMA begins to ravel. The results from the strain measurements and for the cores pulled from the test section indicate that the HMA ravels at a faster rate along the joints, where there is greater access for water to enter the pavement structure. The L/S joint is the most difficult to keep sealed, and therefore, the HMA along this joint was found to be more susceptible to stripping and raveling. Consideration should be given to sealing the joints to limit water from coming into contact with the HMA layer.

During the original construction of the HMA pavement it is more difficult to achieve the desired compaction along the longitudinal joint. Insufficient compaction results in a lower-quality HMA, resulting in reduced support and higher strains in the UTW overlay. The quality of the HMA along the longitudinal edge should be evaluated before it is determined whether UTW is an appropriate rehabilitation alternative.

During evaluations of whether UTW is a viable rehabilitation alternative, cores should be pulled from the pavement to determine if the asphalt is stripping and if the asphalt layer has an adequate thickness. UTW can be successfully placed on as little as 76 mm (3 in.) of asphalt, if the quality of the asphalt is good. The cores should also reveal if the asphalt layer is of uniform thickness and whether stripping and raveling have occurred. If these conditions exist, UTW is a good option for use in the rehabilitation of asphalt pavements.

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