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# The Construction and Performance of Ultra-thin Whitetopping Intersections on US-169

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The Minnesota Department of Transportation (Mn/DOT) constructed an ultra-thin whitetopping project at three intersections on US-169 at Elk River to gain more experience in both the design and performance of ultra-thin whitetopping (UTW). All concrete mixes contained either polypropylene or polyolefin fibers. The compressive strength, flexural strength, Poisson's ratio and elastic modulus were measured for these mixes and the results are provided. Distinct cracking patterns developed within each test section. The strain measurements emphasize the importance of the support provided by the hot mix asphalt (HMA) layer. A reduction in this support occurs when the temperature of the HMA is increased or when the HMA begins to ravel. Cores should be pulled from the pavement when evaluating whether UTW is a viable rehabilitation alternative 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 the asphalt layer is of uniform thickness and stripping/raveling has not occurred. If these conditions exist, UTW is a good option for rehabilitating asphalt pavements.


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# The Construction and Performance of Ultra-thin Whitetopping Intersections on US-169

**State Project No. 7106-60**

# Final Report

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# EXECUTIVE SUMMARY

The Minnesota Department of Transportation (Mn/DOT) constructed an ultra-thin whitetopping project at three intersections on US-169 at Elk River to gain more experience in both the design and performance of ultra-thin whitetopping (UTW). A brief description of the Mn/DOT's history in the area of whitetopping is presented below followed by a detailed description of the construction of the instrumented whitetopping test sections on US-169. All concrete mixes contained either polypropylene or polyolefin fibers. The compressive strength, flexural strength, Poisson's ratio and elastic modulus were measured for these mixes and the results are provided. Distinct cracking patterns developed within each test section. The UTW test sections with a 1.2-m x 1.2-m (4-ft x 4-ft) joint pattern included corner breaks and transverse cracks. Corner breaks were the primary distress in the test section with a 1.8-m x 1.8-m (6-ft x 6-ft), although very little cracking was exhibited. The strain measurements emphasize the importance of the support provided by the hot mix asphalt (HMA) layer. A reduction in this support occurs when the temperature of the HMA is increased or when the HMA begins to ravel. Cores should be pulled from the pavement when evaluating whether UTW is a viable rehabilitation alternative 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 the asphalt layer is of uniform thickness and stripping/raveling has not occurred. If these conditions exist, UTW is a good option for rehabilitating asphalt pavements.

# Chapter 1 Introduction

Whitetopping refers to placing a thin concrete overlay directly on top of an existing distressed HMA pavement. A concrete overlay ranging between 50 to 100 mm (2 to 4 in) thick is commonly referred to as ultra-thin 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/warping and bending stresses. Typical applications would include low to medium volume pavements where rutting, washboarding or shoving are present; such as intersections, bus stops, airport aprons, taxiways or parking lots.

The Minnesota Department of Transportation (Mn/DOT) has constructed only a few ultra-thin whitetoppings to date. The Mn/DOT constructed ultra-thin whitetopping test sections at three consecutive intersections on US-169 in Elk River to gain more experience in both the design and performance ultra-thin whitetopping. A map showing the location of the project is provided in Figure 1.1. 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.

The Jackson Street intersection is the intersection furthest north. School Street and Main Street cross US-169 directly south of Jackson Street. See figure 2. All three intersections have traffic signals. The speed limit on US-169 changes from 89 kph (55 mph) to 72 kph (45 mph) just north of Jackson Street as the traffic approaches the city of Elk River. The traffic light at Jackson Street is the first in a series of traffic lights. 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. The trucks rapidly reduce speed as they approach the first traffic signal at Jackson Street. The speed of the traffic is significantly reduced by the time it approaches the third traffic signal at the Main Street intersection.

1 mile = 1.609 kilometers

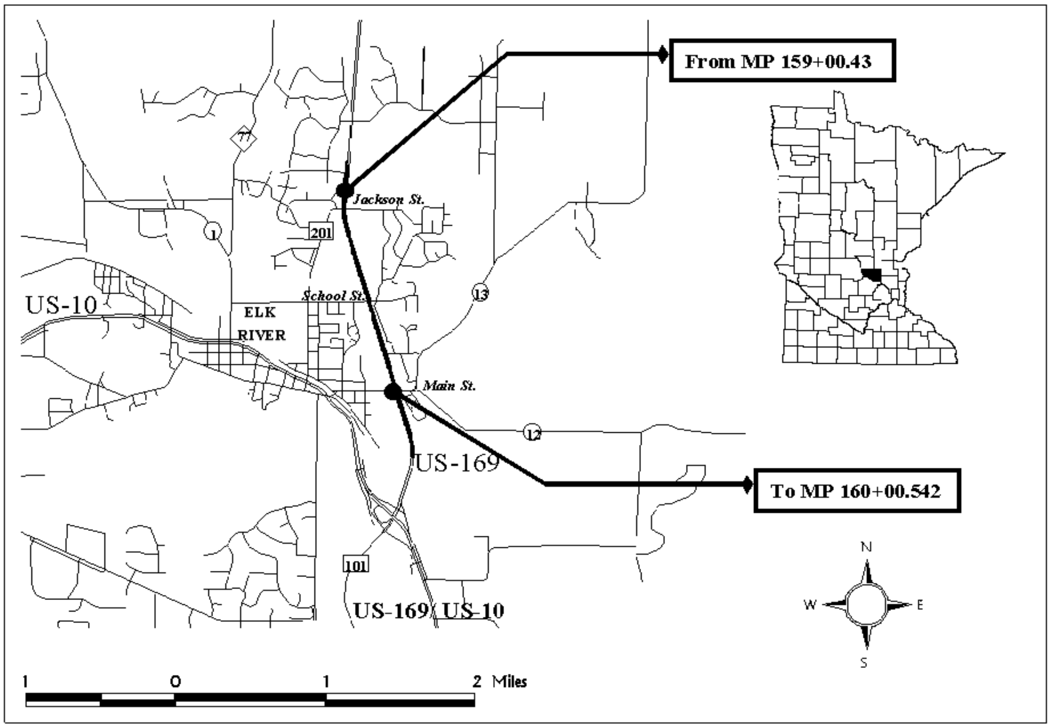


Figure 1.1. US-169 project location.

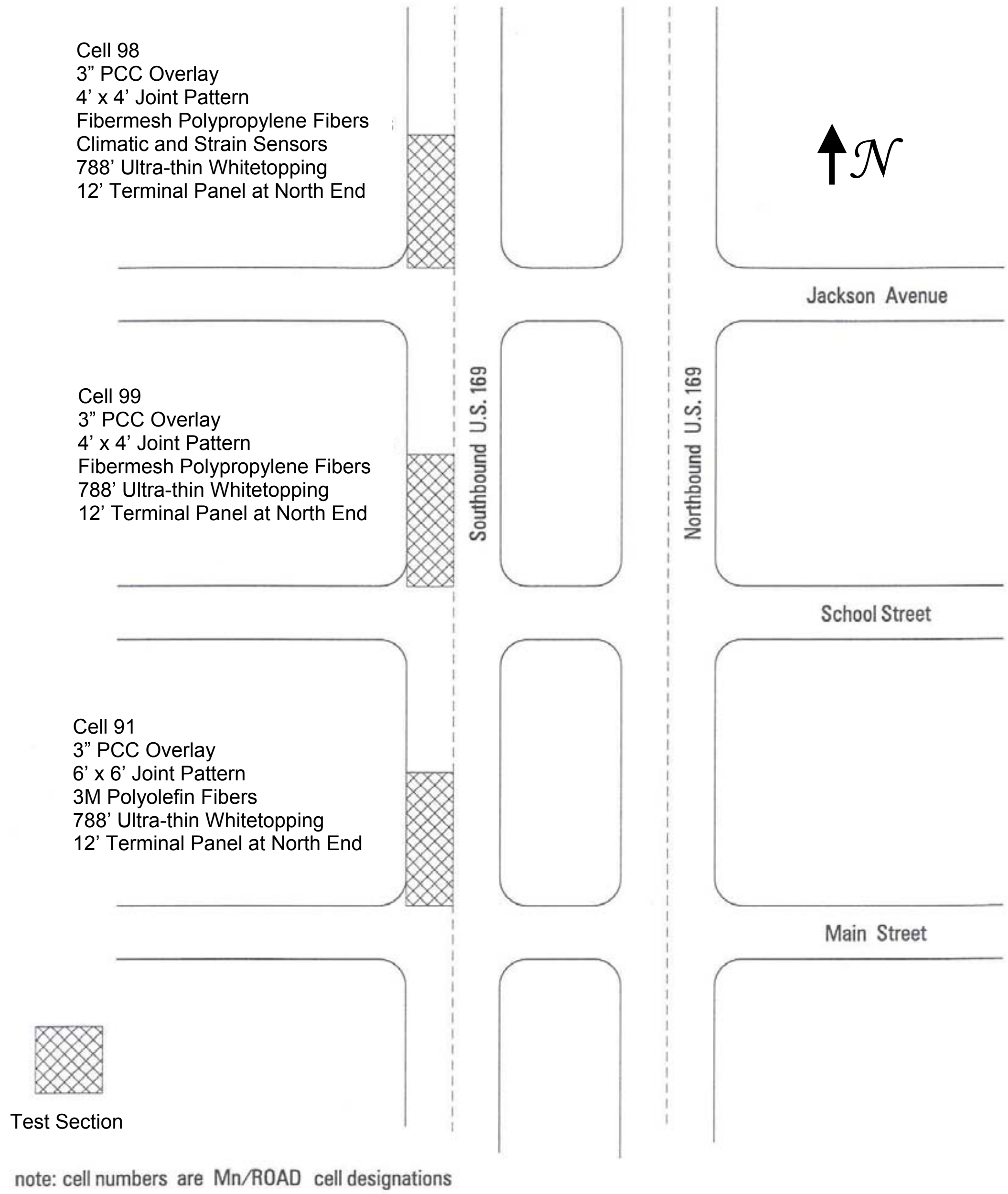


**Figure 1.2. US-169 project layout.**

# Chapter 2 Pre-existing Pavement Structure

Roadway history files indicate the original pavement was constructed in 1961 on a sandy subgrade and consisted of a 100-mm (4-in) HMA surface on 125 mm (5 in) of Class 5 aggregate base and 150 mm (6 in) of Class 4 aggregate base. In 1991, 50 mm (2 in) of HMA was milled and the pavement was overlayed with 40 mm (1.5 in) of HMA. The roadway history files may be incomplete since the average HMA thickness based on a total of ten cores pulled April 8, 1997 between roadway post 159.080 and 160.367 was 160 mm (6.25 in).

A distress survey was performed on each 244-m (800-ft) section prior to the construction of the overlay. Transverse joints were sawed into the HMA pavement approximately every 9 m (30 ft). The average transverse crack/joint spacing at the Jackson Street, School Street and Main Street test sections was 5-m (17 ft), 6-m (20-ft) and 7-m (22-ft), respectively. The driving lane of the Jackson Street test section contained 180 m (590 ft) of transverse cracks/joints and the School and Main Street test sections contained 150 m (490 ft) and 160 m (530 ft), respectively. The cracks were all low to medium severity. The HMA was raveled in areas, especially along the outer edge. Severe rutting (greater than 32 mm (1.25 in)) and shoving was also present prior to milling as a result of heavy trucks stopping and starting at each intersection. See figure 3. A copy of the distress survey has been included in appendix A.

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 prior to the concrete overlay. The pavement substructure was relatively dry at this time, as indicated by the precipitation data provided in figure 4. Figure 4 is a plot of the precipitation measured by

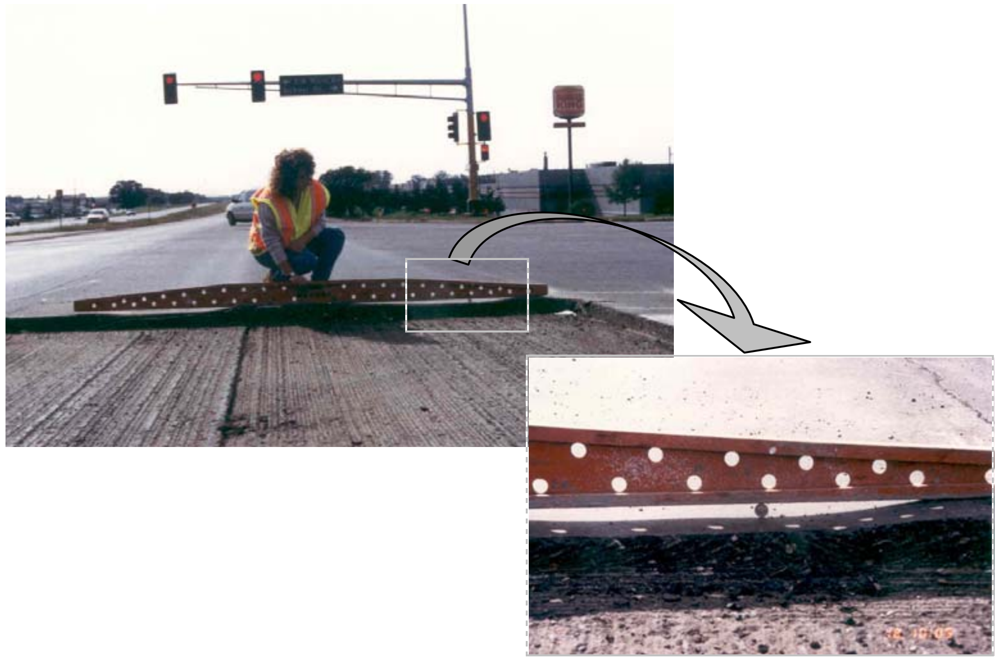

**Figure 2.1. Ruts at the Jackson Street intersection prior to the UTW overlay.**

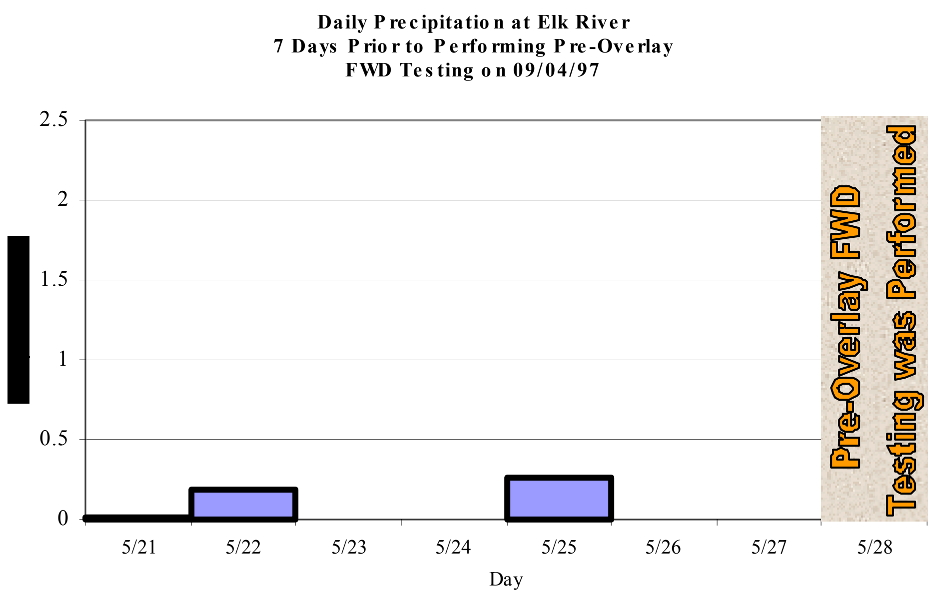


**Figure 2.2. Precipitation measured over the seven days prior to FWD testing.**

a weather station in the area of the project for the 7 days prior to performing the pre-overlay FWD testing. The graph reveals less than 25 mm (0.5 in) of precipitation was measured over the seven days prior to testing. The precipitation data was collected at the weather station at the Mn/ROAD research facility. The Mn/ROAD research facility is approximately 15 km (10 m) from the Elk River UTW test sections.

The average of three deflection measurements for a 40-kN (9-kip) load was plotted for each sensor and provided in figures 5 thru 7. The deflections measured at the Jackson Street intersection were lower than those measured at School Street and Main Street All FWD testing was performed on the same day. Testing began on the Jackson Street test section with the pavement surface temperature ranging between 20ºC and 23ºC (68ºF and 73ºF). The School Street section was tested next followed by the Main Street section. The surface temperature ranged between 22ºC and 24ºC (72ºF and 76ºF) and 24ºC and 27ºC (75ºF and 80ºF) for the School Street and Main Street sections, respectively. The deflections closer to the load increased from test section to test section with increasing pavement surface temperature. The deflections measured further away from the load are similar between test sections indicating the structural support provided by the underlying layers is similar between test sections.

The deflection measured directly under the load plate (D0) provides an indication of the stiffness of the pavement structure. The average of three normalized deflections measured for a 40-kN (9-kip) load in each of the test sections are provided in figure 8. The stiffness of the pavement is sensitive to the temperature of the asphalt so all

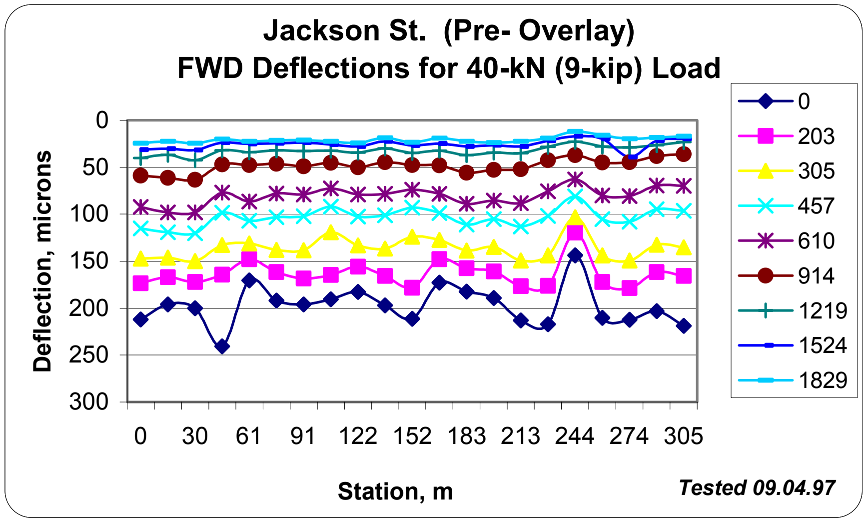


**Figure 2.3. Deflections measured at the Jackson Street test section prior to the overlay.**

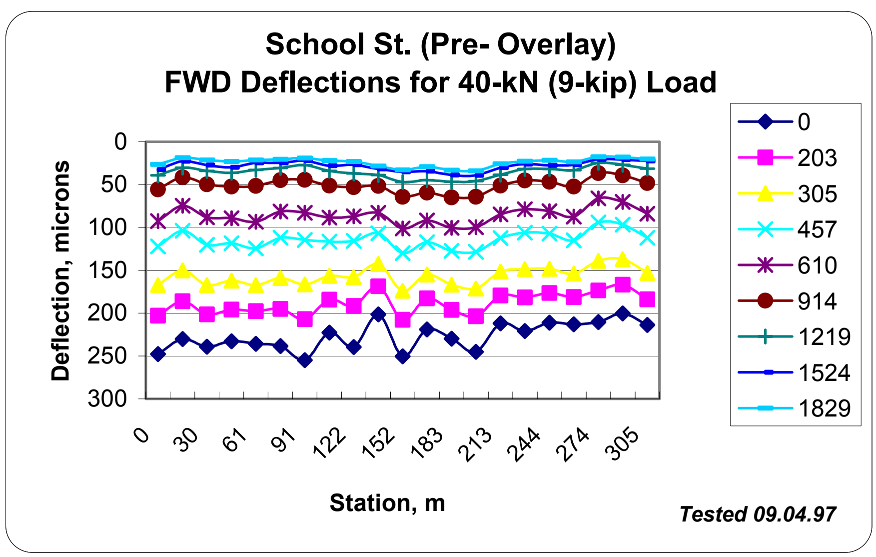


*Figure 2.4. Deflections measured at the School Street test section prior to the overlay.*

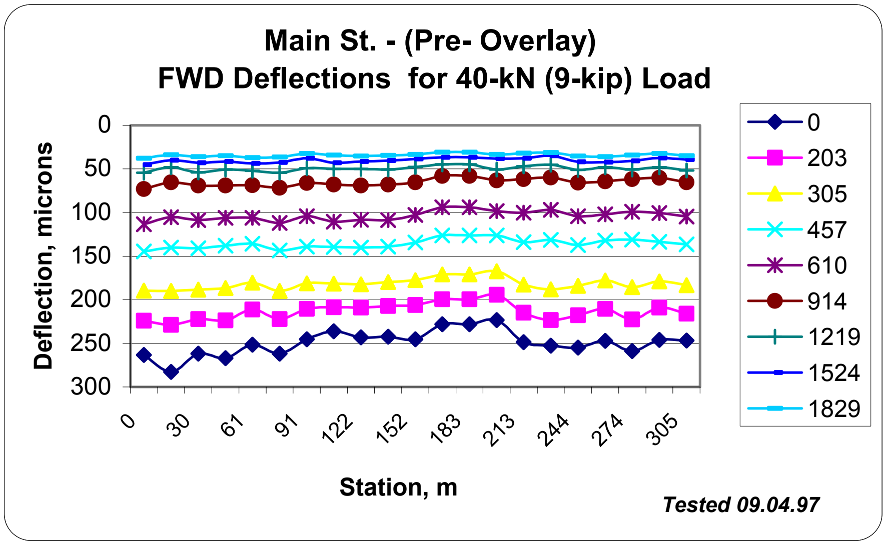


*Figure 2.5. Deflections measured at the Main Street test section prior to the overlay.*

measured deflections were adjusted to the deflections expected at a mid-depth asphalt temperature of 20$^o$C. These adjustments were made using the procedure presented by Lukanen, et al, 2000. The mid-depth asphalt temperature at the time of testing was estimated using BELLS3 (Lukanen, et al, 2000). The deflections are similar for all test sections but are slightly lower at the north end of the Jackson Street test section indicating the pavement structure might be slightly stiffer in this area.

The AREA basin factor was calculated for the deflection data (Hoffman and Thompson, 1981). AREA is derived from the area of the deflection basin curve normalized with respect to the deflection recorded directly under the load plate. The AREA factor represents the ratio of the stiffness of the pavement and the stiffness of the subgrade. The AREA factor was calculated using the average of the three deflections normalized to a 40-kN (9-kip) load. The mid-depth asphalt temperature during testing was estimated using BELLS3 (Lukanen, et al, 2000). The AREA basin factors were adjusted to a mid-depth asphalt reference temperature of 20$^o$C using the procedure presented by Lukanen, et al, 2000. AREA basin factors calculated for each test section are provided in figure 9. The ratio between the stiffness of the pavement and the stiffness of the subgrade appears to be relatively constant within and between each test section. Although, AREAs 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 reflected in figure 8 in this region might be attributed to a stiffer pavement.

An attempt was made to backcalculate the resilient modulus of each layer assuming each layer consisted of a linear elastic homogeneous material. It was readily apparent that the response of various layers was nonlinear. The nonlinearity in the upper layers was in the form of stress stiffening and possibly stress-softening in the lower layers. The analysis did indicate the subgrade is very strong, as would be expected since

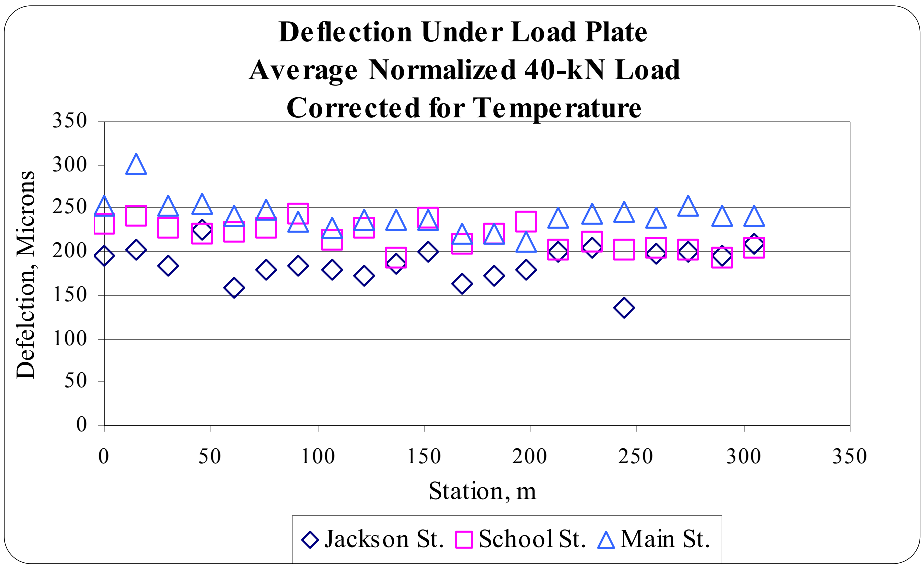


**Figure 2.6. FWD deflections under load plate measured for the HMA pavement.**

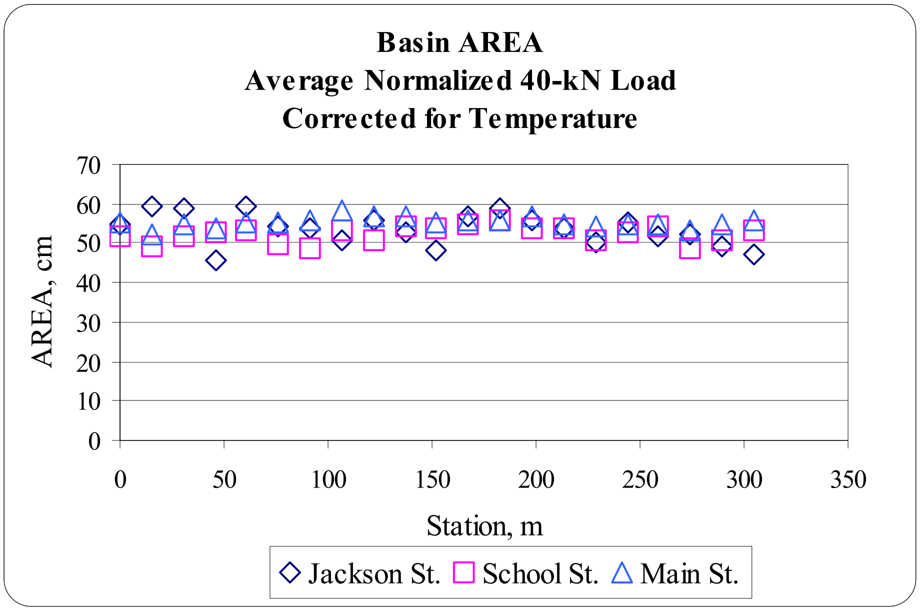


**Figure 2.7. Basin AREAs calculated for each test section prior to the overlay.**

the subgrade consists of a sandy gravel. The base was constructed of Class 5 and Class 4 aggregates containing a large amount of fine material so stiffness of this material is most likely not as high as is desired. The deflection data also indicated the presence of stripping in locations. This was verified when forensic cores were pulled from the ultra-thin whitetopping just prior to reconstruction, as will be discussed below.

# Chapter 3 Description of Test Sections

The ultra-thin whitetopping was only constructed 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 ultra-thin whitetopping sections contained polypropylene fibers and the School Street intersection contained polyolefin fibers. The Jackson and Main Street test sections had 1.2-m x 1.2-m (4-ft x 4-ft) panels. The School Street intersection had 1.8-m x 1.8-m (6-ft x 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). The purpose of the thicker section was to reduce the damage that would occur as heavy trucks come off the HMA pavement onto the ultra-thin whitetopping. A description of each test section is provided in table 1 and figure 2. Temperature (type-T thermocouples) and dynamic (Tokyo Sokki PML-60) and static (Geokon VCE 4200 vibrating wire strain gages) strain sensors were installed approximately 37 m (120 ft) north of the Jackson Street intersection. The sensor layout is provided in appendix B.

**Table 3.1. Summary of US-169 whitetopping test sections.**

| *Test Cell Description* | *Instrumentation* | *No. of Sensors* |
|---|---|---|
| **Jackson Street intersection:**<br>75 mm – 1.2-m x 1.2-m Panels<br>(3 in – 4-ft x 4-ft)<br>Polypropylene Fibers | Dynamic Strain<br>Static Strain<br>Thermocouple | 32<br>4<br>14 |
| **School Street intersection:**<br>75 mm – 1.2-m x 1.2-m Panels<br>(3 in – 4-ft x 4-ft)<br>Polypropylene Fibers | -None- | -- |
| **Main Street intersection:**<br>75 mm – 1.8-m x 1.8-m Panels<br>(3 in – 6-ft x 6-ft)<br>Polyolefin Fibers | -None- | -- |
| | **Total** | **50** |

## Mix Designs

The concrete for the project required an air content of 6.5 percent ± 1.5 percent and a 3-day, 2.8 MPa (400 psi) flexural strength, so that the overlays could be opened to traffic in three days. Two different mixtures were used. The Jackson Street and Main Street intersections contained concrete with polypropylene fibers and the School Street intersection had polyolefin fibers. The polyolefin mixes contained 14.8 kg/m$^3$ (25 lbs/yd$^3$) of fibers and the polypropylene mixes contained 1.8 kg/m$^3$ (3 lbs/yd$^3$) of fibers. A maximum 0.44 water to cementitious ratio (w/cm) was specified for the polyolefin mixtures and 0.40 for the polypropylene mixtures. A water reducer was also used in both mixtures. The mixture designs used on US-169 are provided in table 2. Aggregate gradations and mix material sources are provided in appendix C. A central mix plant was used for batching the concrete. The concrete was batched in 8 m$^3$ (10 yd$^3$) loads and mixed for 5 minutes before being loaded into a truck. The concrete was agitated in the truck while being transported to the site, which was approximately 15 minutes from the plant. Plant problems resulted in a higher than desired w/cm ratio for the polypropylene mix. The problem was rectified by the time the last intersection, which contained the polyolefin fibers, was paved.

**Table 3.2. Mix designs used on US-169, in Elk River.**

| | *Concrete with Polypropylene Fibers* | *Concrete with Polyolefin Fibers* |
|---|---|---|
| **Water/Cementitious Ratio** | 0.43 | 0.37 |
| **Cement, kg/m$^3$ (lbs/yd$^3$)** | 267 (450) | 327(550) |
| **Class C Fly Ash, kg/m$^3$ (lbs/yd$^3$)** | 71(120) | 59(100) |
| **Fine Aggregate, kg/m$^3$ (lbs/yd$^3$)** | 761(1287) | 761(1287) |
| **CA (19 mm minus), kg/m$^3$ (lbs/yd$^3$)** | 918(1552) | 887(1500) |
| **CA (10 mm minus), kg/m$^3$ (lbs/yd$^3$)** | 164(277) | 164(277) |
| **Fiber Content, kg/m$^3$ (lbs/yd$^3$)** | 1.8(3) | 14.8(25) |
| **Measured Air, %** | 6 | 6 |
| **Measured Slump, mm (in.)** | 57(2.25) | 50(2) |

## 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 macro-texture from the ridges milled into the surface and the freshly fractured aggregate surfaces that were exposed. After milling, the underlying HMA at the Jackson Street intersection appeared to be more severely raveled than the other two intersections, especially along the longitudinal seams between the roadway and the shoulder. Some of the areas were so severely raveled that the pressure imposed when air-blasting would flake off pieces of the HMA from the pavement structure. No pre-overlay repairs were performed. The temperature and strain sensors were installed at the Jackson intersection and the Jackson intersection was paved on September 17. The first load of concrete arrived at 14:45 and last load arrived at 18:30. The remaining two intersections were paved on the following day. The first load of concrete arrived at 7:40 at the School Street intersection and last load arrived at 11:45. Paving of the Main Street intersection began at 12:30 and the last load of concrete was placed at 16:30. Concrete placement was accomplished using a clary screed. The concrete was placed directly on the milled surface without a tack coat or whitewash. The milled surface was also relatively dry when the concrete was placed.

All finishing and surface texture work was performed by hand. An extra step was added to the finishing process of the polyolefin concrete on US-169. A special finishing tool was used to roll the fibers on the pavement surface down into the fresh concrete. This helped keep the fibers from rising to the surface when final finishing with the bull-floats was performed. A curing compound was applied at the rate of 4.6 $m^2/L$ (18 $yd^2/gal$) shortly after tining. Wet burlene was placed over the whitetopping sections after paving. The cooler nighttime temperatures increased the concrete set time so the contractor could not begin sawing the joints until the morning after paving. The burlene blankets were removed during sawing and then placed back on after the sawing was completed. The joints were sealed with hot pour. On the morning of September 22, the burlene blankets were removed and the pavement was opened to traffic. Unlike most

Mn/DOT concrete paving projects, this project did not include ride quality requirements. The resulting ride over these intersections was still relatively smooth.

**<u>Hardened Concrete Properties</u>**

Beams and cylinders were cast for measuring flexural and compressive strengths. Six 152-mm x 152-mm x 533-mm (6-in x 6-in x 21-in) beams and thirteen 102-mm x 203-mm (4-in x 8-in) cylinders were cast, stripped the following day and then cured in an environmental room according to ASTM C31. The beams were broken after 28 days using a field beam breaker. The average strengths are provided in table 3. Six cylinders were broke after 14 days and seven cylinders after 28 days. The average compressive strengths are provided in table 4.

**Table 3.3 Flexural strengths for US-169.**

| | ***Concrete with Polypropylene Fibers*** *(Jackson Street intersection) Mpa (psi)* | ***Concrete with Polyolefin Fibers*** *(Main Street intersection) Mpa (psi)* |
|---|---|---|
| *28-Day* | 4.1 (590) | 3.9(570) |

**Table 3. 4 Compressive strengths for US-169.**

| | ***Concrete with Polypropylene Fibers*** *(Jackson Street intersection) Mpa (psi)* | ***Concrete with Polypropylene Fibers*** *(School Street intersection) Mpa (psi)* | ***Concrete with Polyolefin Fibers*** *(Main Street intersection) Mpa (psi)* |
|---|---|---|---|
| *14-Day* | 33.8 (4900) | 33.8 (4900) | 30.3(4400) |
| *28-Day* | 37.2 (5400) | 40.7 (5900) | 36.6 (5300) |

The flexural strengths for both the polypropylene and polyolefin mixes were similar. The polypropylene concrete had slightly higher compressive strengths even though the w/cm ratio for the polyolefin mixture was lower. Increasing the fiber content in the polyolefin mixture by eight times that of the polypropylene mixture contributed to the lower strength of the polyolefin concrete. Both mixes met Mn/DOT's flexural and compressive strength requirements.

**Traffic**

The test sections on US-169 were in service between September 1997 and September 1999. During this period, the sections accumulated approximately 670,000 equivalent single axle loads (ESALS) (assuming a 152-mm (6-in) portland cement concrete pavement and a terminal serviceability of 2.5). The one-way average annual daily traffic (AADT) was 16,000 in 1997 for this section of roadway with 8 percent being trucks. The AADT grew to 17,000 by 1999. Forty-nine percent of these trucks are categorized as five-axle semis.

# Chapter 4 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 each of the test sections as the vehicles came off from the HMA pavement onto the ultra-thin. The most heavily distressed area in each of the test sections was just prior to the intersection. The change in vehicle speed is the greatest in this location as vehicles accelerate and decelerate when the traffic light changes.

Cracks observed in the ultra-thin whitetopping test sections with 1.2-m x 1.2-m (4-ft x 4-ft) joint pattern include 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 1/3 of the length of the panel. This crack pattern is shown in figure 10 and 11. The photo in figure 7 was taken of the Jackson Street test section on March 30, 1998 and the photo in figure 8 was taken July 20, 1999. A copy of the distress survey performed on March 30, 1998 for the Jackson Street test section is provided in appendix D.

Comparing the pre-overlay distress survey to the distress surveys performed after the overlay was constructed revealed none of the transverse joints or cracks in the HMA reflected into the overlay for any of the test sections. Reflective cracks did develop in the 76-mm and 102-mm (3-in and 4-in) overlays constructed on I-94. The same joint patterns used on US-169 were also constructed on I-94. The difference in the performance can be attributed to the fact that the UTW on US-169 was placed on 76 mm (3 in) of HMA exhibiting signs of raveling and the UTW on I-94 was constructed on

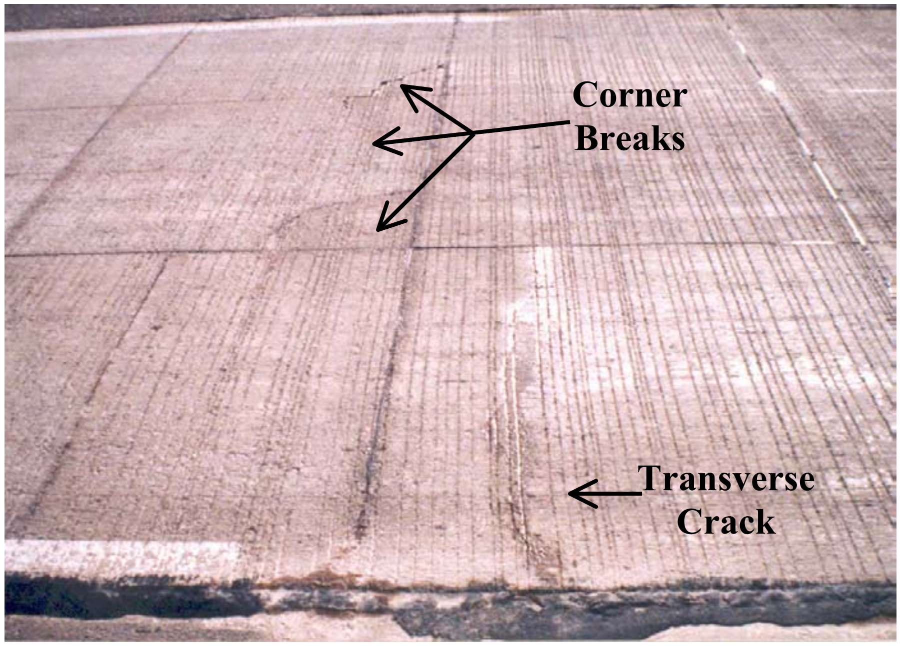


**Figure 4.1. Transverse crack and corner breaks in the Jackson Street test section (photo was taken on 03.30.98).**

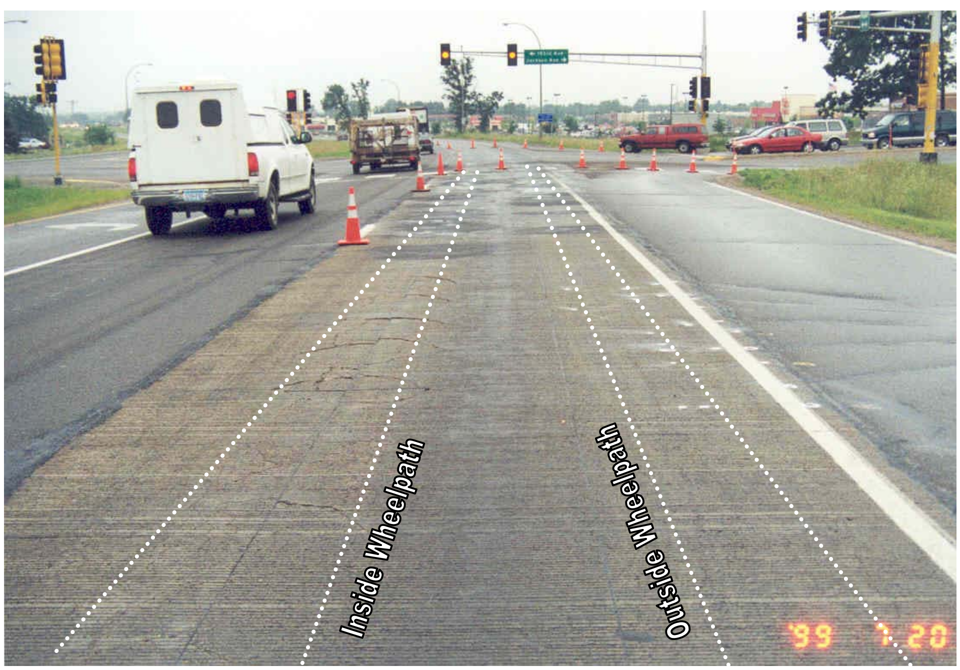


**Figure 4.2. Corner breaks in the inside wheelpath at the Jackson Street test section (photo was taken on 07.20.99).**

254 mm (10 in) or more of quality HMA. This resulted in a higher bond strength and 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 Main Street test section was constructed using a 1.8-m x 1.8-m (6-ft x 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 lane/shoulder joint and intersect the transverse joint in the wheelpath. A few corner breaks also developed in the inside panels. Again, the corner break typically intersected the transverse joint in the wheelpath but then intersects the longitudinal joint separating the two panels. The corner breaks exhibited in both the inside and outside panels intersect the longitudinal joint nearest to each wheelpath. See figure 12. A distress survey was performed on this test section on September 15, 1999 and has been included in appendix D.

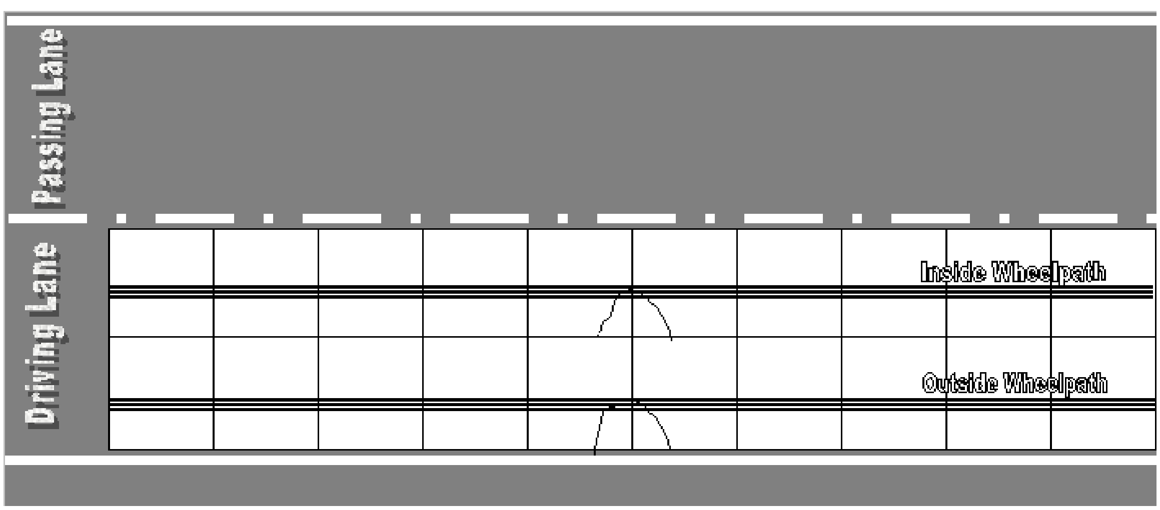


**Figure 4.3 Typical distress patterns that developed in the Main Street test section.**

The number and the severity of the distresses exhibited in the Jackson Street test section were higher than School or Main Street The number of distressed panels in the Jackson Street test section was approximately twice as high as the number of distressed panels at the School Street intersection and four times as high at the Main Street intersection. The difference between the performances of the School Street and Jackson Street test sections is somewhat surprising because the overlay design is the same. There are several possible explanations. First, raveling of the HMA at the Jackson Street intersection was greater than the at the School Street intersection. Also, the speed limit on US-169 changes from 89 kph (55 mph) to 72 kph (45 mph) just north of Jackson Street and the traffic light at Jackson Street is the first in a series of traffic lights with the School Street intersection following the Jackson Street intersection. The commercial trucks traveling southbound on US-169 are rapidly reducing speed as they approach the first traffic signal at Jackson Street and the speed of the traffic is significantly reduced by the second intersection at School Street Therefore, the dynamic stresses on the School Street test section are most likely significantly lower than on the Jackson Street test section.

The Main Street test section performed significantly better than the Jackson and School Street intersection because the longitudinal joint does not lie in the inside wheelpath for a 1.8-m x 1.8-m (6-ft x 6-ft) joint pattern. This significantly reduces the edge and corner stresses.

After cracking began to occur, the maintenance crew in District 3 tried to repair the cracked panels using a blowpatch machine. The blowpatch machine blows compressed air to clean the pavement surface. An asphalt emulsion is sprayed and then a combination of emulsion and aggregate is discharged. The aggregate has a top size of 6 to 10 mm (0.25 to 0.375 in). This resulted in an increase in ride roughness and is not recommended for UTW. Areas with blowpatches can be seen in the photos of the test sections provided in appendix D.

## Falling Weight Deflectometer Data

FWD testing was performed at various times of the year in 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 have been provided in figure 13. The deflection basins produced by approximately a 40-kN (9-kip) load were normalized to a 40-kN (9-kip) load and averaged. A graph of each measured deflection basin measured at the three test sections is provided in appendix E.

Daily precipitation data was collected at the Mn/ROAD weather station for 1998 and 1999. See figures E.1 and E.2. Table 5 summarizes the precipitation accumulated within the first, third and seventh days prior to performing the FWD testing on the Elk River test sections.

**Table 4.1 Summary of precipitation seven days prior to FWD testing.**

| *Test Date* | *Cumulative Precipitation Prior to FWD testing, mm* | | |
|---|---|---|---|
| | *Day 1* | *Day 3* | *Day 7* |
| 3/30/1998 | 21 | 69 | 72 |
| 4/9/1998 | 0 | 14 | 14 |
| 7/1/19998 | 0 | 35 | 60 |
| 10/9/1998 | 0 | 0 | 7 |
| 2/4/1999 | 0 | 0 | 5 |
| 7/20/1999 | 0 | 0 | 11 |

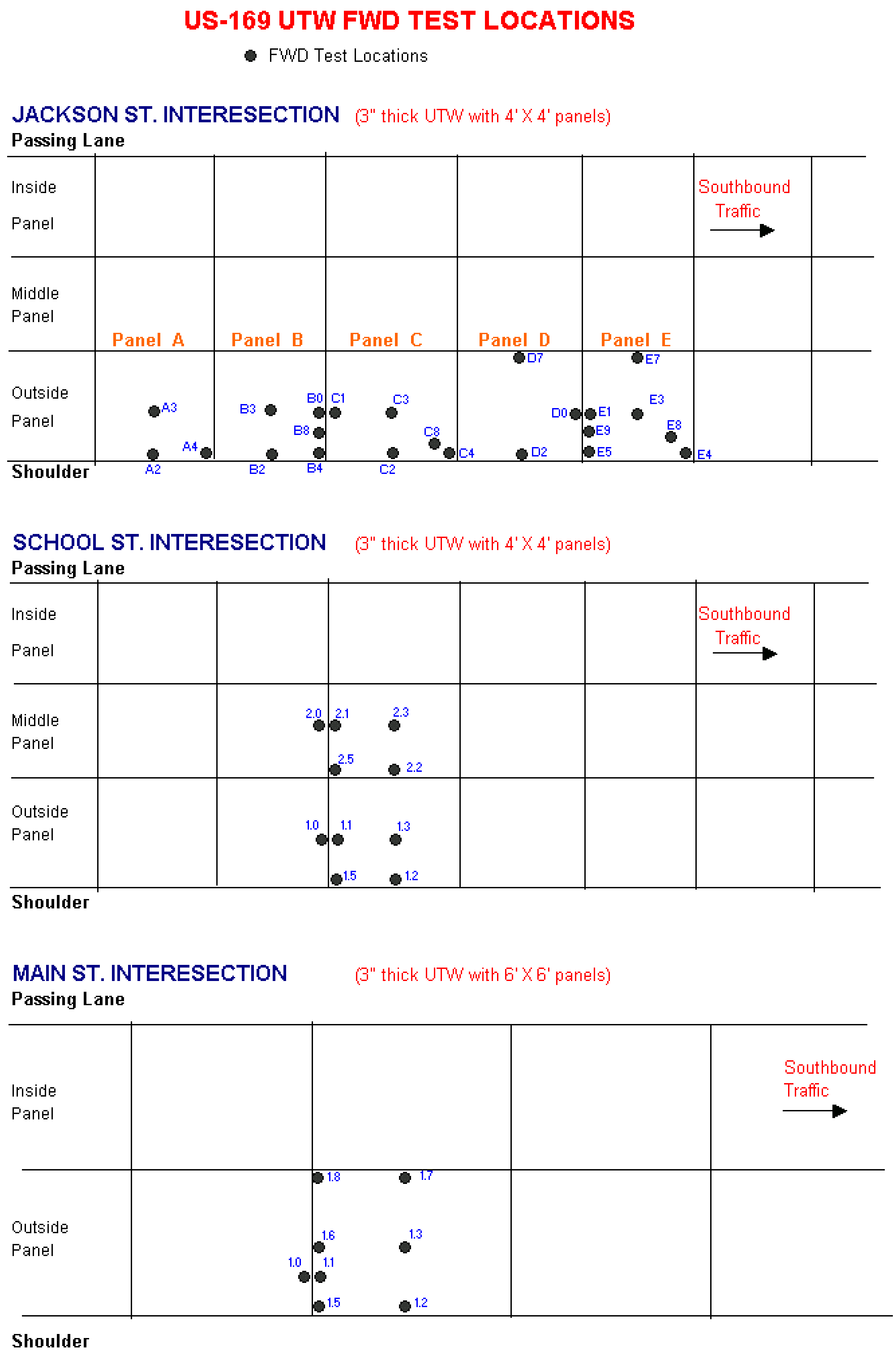


**Figure 4.4 US-169 UTW FWD test locations at the Jackson, School, and Main street intersections.**

The location of the transverse joint with respect to the location of the applied FWD load is indicated in each graph provided in appendix E when FWD testing was performed within 1810 mm (72 in) of a joint. A discontinuity typically appeared in the deflection basin at the location of the transverse joint if the applied load is within in 600 mm (24 in) of the joint at Jackson and School Street An exception to this occurred when FWD testing was first performed in March and April of 1998 approximately 5 months after the test sections were constructed. The smooth deflection basins obtained during this time indicate most joints did not crack prior to March 1998. The deflection data collected during July 1998 indicates that most of the joints did crack sometime between March and July of 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 adjacent to the transverse joint. Therefore, it was not possible to estimate when the joints cracked based on FWD data for the Main Street test section.

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 6 times higher. The highest deflections were typically measured in the summer when the asphalt is less stiff, as would be expected. This trend was more prevalent at the Jackson and School Street intersections and when loading at the edge or in the corner where the response of the slab is more heavily influenced by changes in support conditions.

The average normalized deflection for a 40-kN (9-kip) FWD load applied in the corner, in the wheelpath on the approach side of the joint, at midpanel adajacent to the inside edge and at the geometric center of the panel is plotted against the mid-depth asphalt temperature at the time of testing. Figure 14 contains the Jackson Street deflection data and figures 15 and 16 contain deflection data for School Street and Main Street, respectively. 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 lane/shoulder joint. The areas with the higher deflections also correspond 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 different panels. This indicates 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. Jackson Street exhibited the least amount of variability between deflections

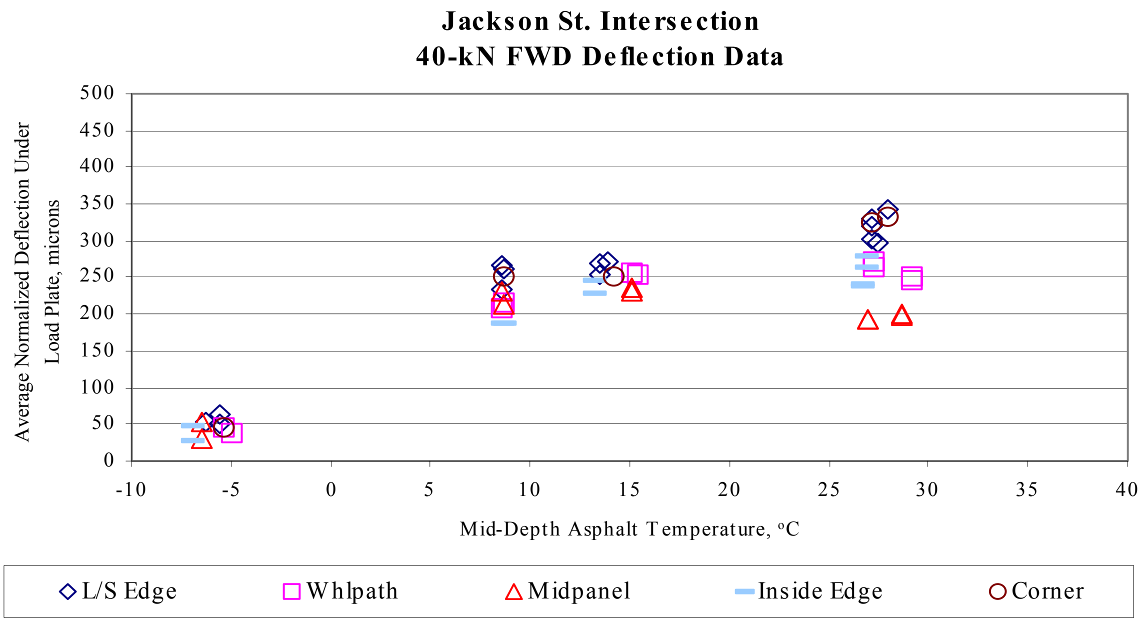


**Figure 4.5 Average normalized deflections measured directly under the load plate for40-kN (9-kip) FWD load at Jackson Street**

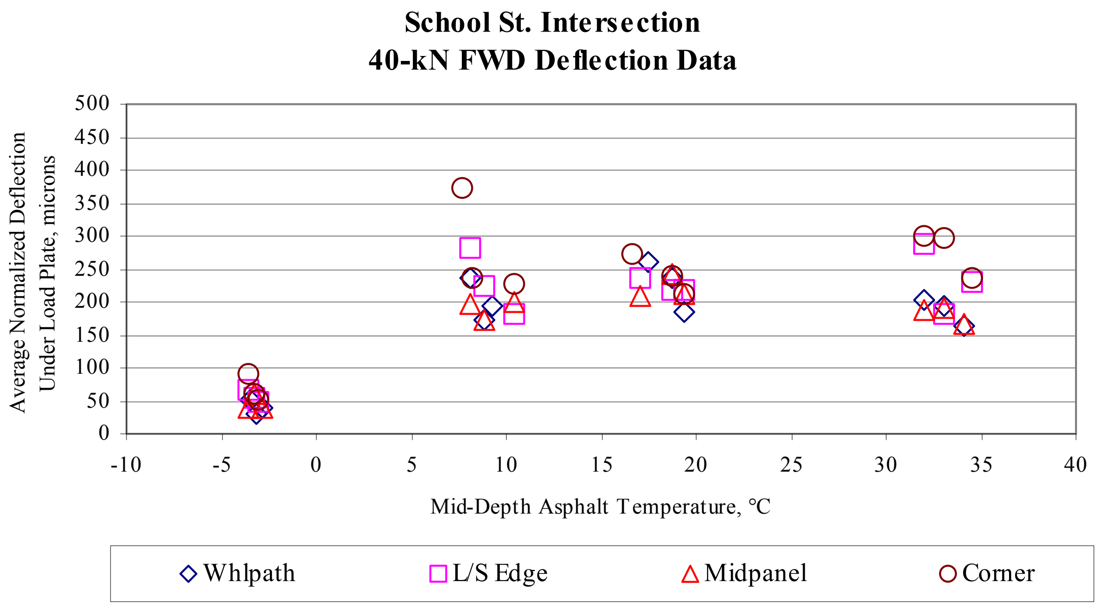


**Figure 4.6 Average normalized deflections measured directly under the load plate for40-kN (9-kip) FWD load at School Street**

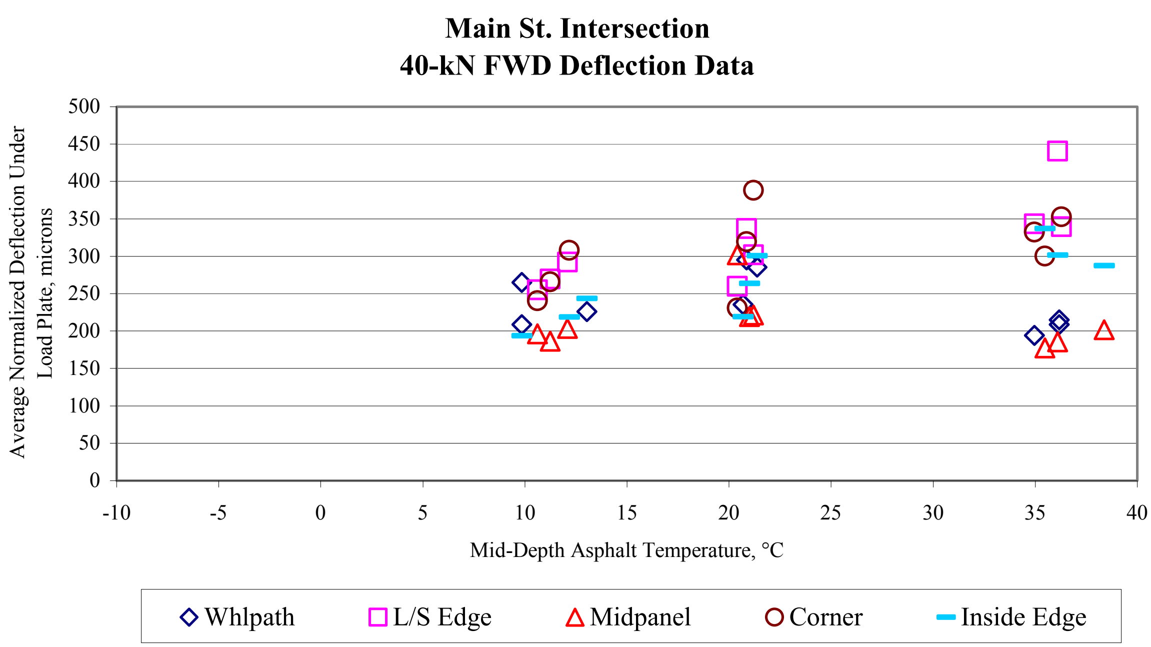


*Figure 4.7 Average normalized deflections measured directly under the load plate for40-kN (9-kip) FWD load at Main Street*

measured at the same location and the same time but different panels. A couple of factors most likely contributed the uniformity of the deflections. First the panels tested at the Jackson Street intersection were all in the same location so support conditions were similar. Also, more effort was put into cleaning the pavement surface at the Jackson Street test locations because sensors were installed. This might have resulted in more uniform bond characteristics and therefore less variability in the measured deflections. The variability between test locations within each test section also increased with increasing asphalt temperatures.

In October 1997, approximately the same time the U.S.-169 test sections were constructed, six other whitetopping sections were constructed on I-94 at the Mn/ROAD research facility. I-94 is a heavily trafficed road with an average daily traffic of approximately 25,000 of which 12 to 13 percent is truck traffic. An interstate highway is not a typically application for UTW but this location offered the opportunity to perform an accelerated test for UTW because design loads comparable for that found at a more traditional UTW site could be accumulated more rapidly. Also, at the Mn/ROAD research facility the interstate traffic is diverted onto an adjacent segment of road once a month to allow researches full access to the test road for data collection. The HMA pavement was in relatively good condition prior to 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. One of the test sections at Mn/ROAD consisted of a 76-mm (3-in) UTW with 1.2-m by 1.2-m (4-ft by 4-ft) panels on 270 mm (10.5 in) of HMA. The Mn/ROAD test sections allow comparisons to be made of the same UTW design on HMA pavements with different structural capacities. Comparisons were made between two other test sections included in the Mn/ROAD whitetopping study and the U.S.-169 test sections. A summary of the design features of these sections is provided in table 6.

*Table 4.2. Summary of Mn/ROAD UTW test sections.*

| ***Test Cell*** | ***Overlay*** | ***AC thickness*** |
|---|---|---|
| 93 | 102 mm – 1.2-m x 1.2-m Panels<br>(4 in – 4-ft x 4-ft) | 241 mm<br>(9.5 in) |

| | | |
|---|---|---|
| | Polypropylene Fibers | |
| 94 | 75 mm – 1.2-m x 1.2-m Panels<br>(3 in – 4-ft x 4-ft)<br>Polypropylene Fibers | 267 mm<br>(10.5 in) |
| 95 | 75 mm – 1.8-m x 1.8-m Panels<br>(3 in – 5-ft x 6-ft)<br>Polyolefin Fibers | 267 mm<br>(10.5 in) |

Comparisons were made between the responses of the different UTW designs at both U.S.-169 and Mn/ROAD. Graphs of the average normalized deflections measured directly under the load plate for a 40-kN (9-kip) FWD load applied at the corner, midpanel, inside longitudinal edge, L/S edge and in the wheelpath at different asphalt temperatures are provided in figures 17-21. The deflections were significantly lower at Mn/ROAD compared to U.S.-169 because the existing asphalt was thicker and less deteriorated prior to the overlay. The condition of the asphalt at Mn/ROAD was uniformly good throughout the project, unlike at Elk River. This resulted in more consistent deflection measurements within each Mn/ROAD 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. This emphasizes the need to ensure the asphalt layer is sufficiently thick before considering UTW as a rehabilitation

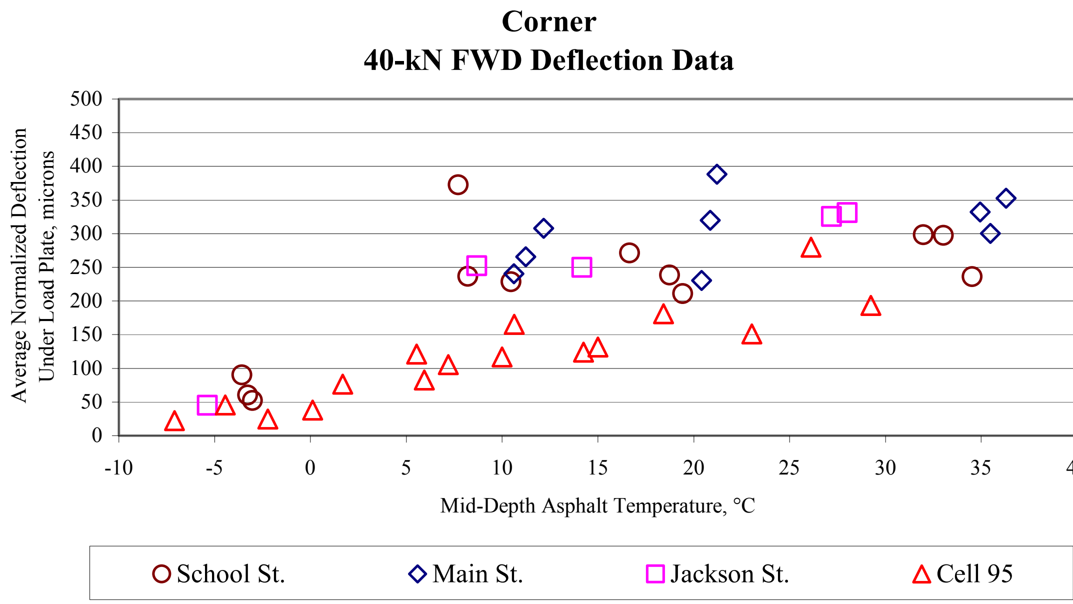


*Figure 4.8 Average normalized deflections measured directly under the load plate for40-kN (9-kip) FWD load in the corner.*

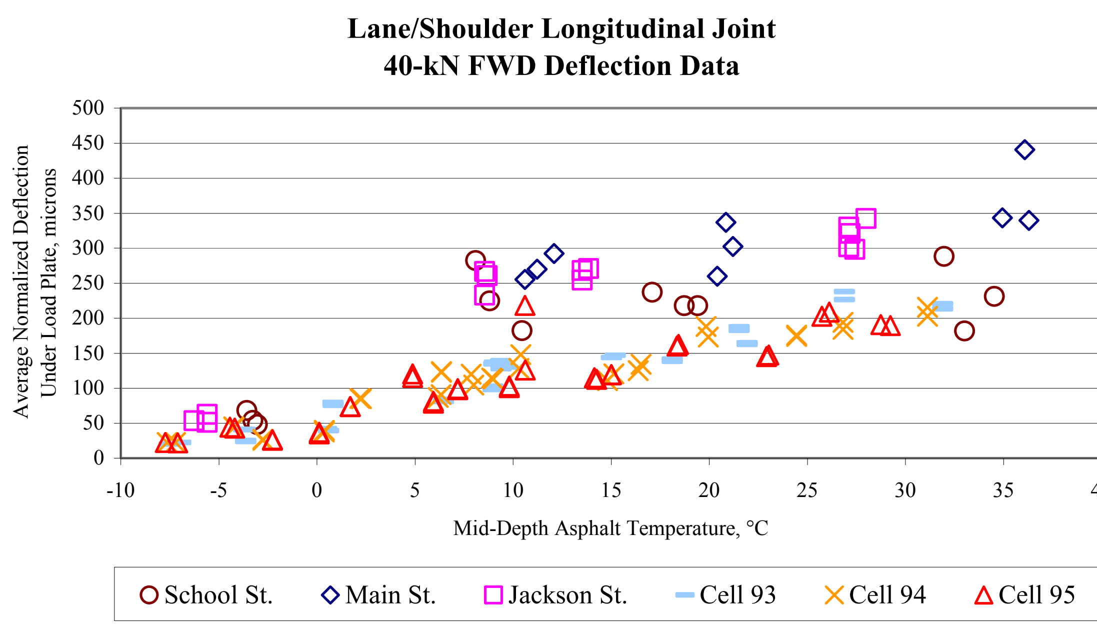


*Figure 4.9 Average normalized deflections measured directly under the load plate for40-kN (9-kip) FWD load along the L/S edge.*

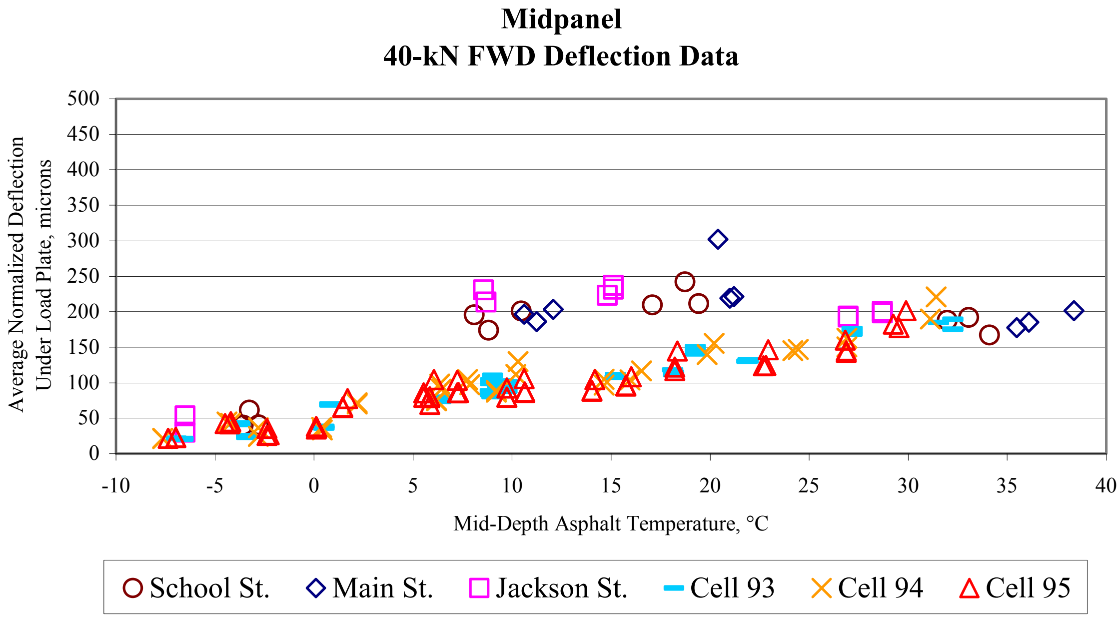


*Figure 4.10 Average normalized deflections measured directly under the load plate for40-kN (9-kip) FWD load at midpanel.*

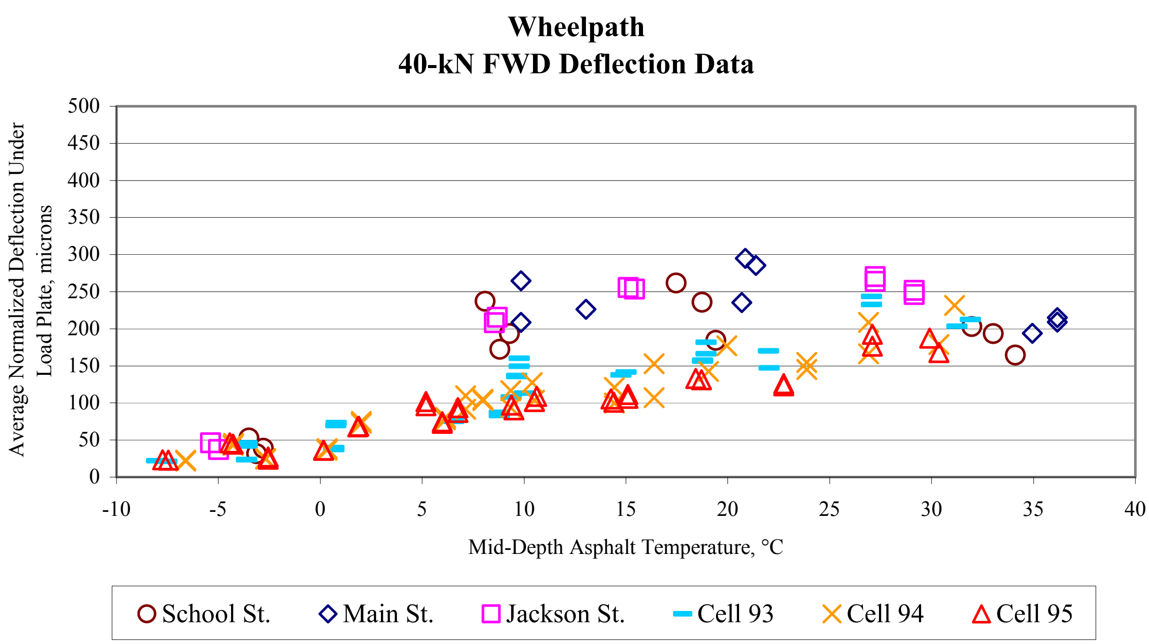

*Figure 4.11 Average normalized deflections measured directly under the load plate for40-kN (9-kip) FWD load in the wheelpath.*

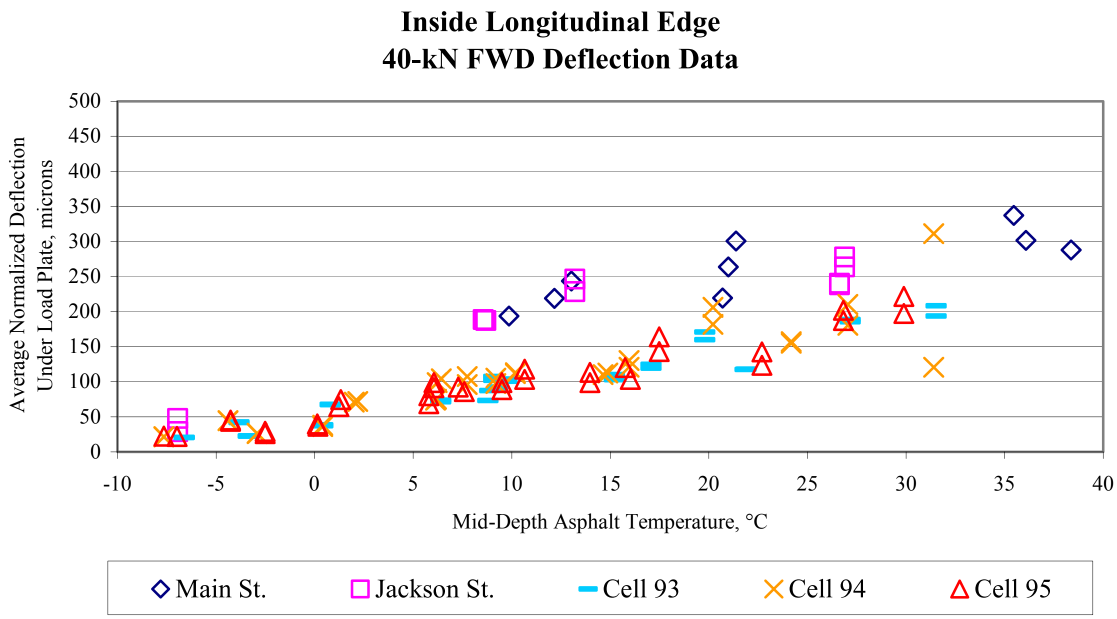


*Figure 4.12 Average normalized deflections measured directly under the load plate for40-kN (9-kip) FWD load along the inside longitudinal joint.*

alternative. The Mn/ROAD test sections not only had lower deflections but the relationship between deflection and temperature was relatively linear compared to the deflections measured at U.S.-169. The Mn/ROAD test sections are still in place after accumulating 4 million ESALS showing that the lower deflections will result an extended pavement life.

**Measured Dynamic Strains**

The Jackson Street test section was instrumented with dynamic strain gages. Strain gages were located at the bottom of the UTW and approximately 25 mm (1 in) from the surface of the overlay at the following locations; in the wheelpath, adjacent to the transverse joint, diagonal in the outside corner, at midpanel adjacent to the land/shoulder longitudinal joint, along the inside longitudinal joint, at midpanel and 305 mm (12 in) from the L/S longitudinal joint adjacent to the transverse joint. Each senor location is replicated accept for the static strain sensors. The sensor locations and orientations are shown in figure B.1 and B.2 and table B.1.

Dynamic strains were measured in conjunction with FWD. The average of three strain measurements resulting from a 40-kN (9-kip) load applied directly over each sensor was plotted against the temperature measured at mid-depth of the HMA layer. The results are provided in figures F.1 through F.9 of appendix E. Positive values represent tensile strains and negative values are compressive strains. Figures F.10 through F.14 depict the temperature profiles throughout the depth of the UTW and HMA that were present while the FWD testing was performed.

The strain measured for each location remained relatively constant when the HMA temperature was greater than 10$^{o}$C (50$^{o}$F). A significant decrease in strain occurred when FWD testing was performed and the HMA temperature was below -5$^{o}$C (23$^{o}$F) as a result of an increase in the modulus of the HMA and the frozen base. The temperature of the HMA was between 25$^{o}$C and 30$^{o}$C (77$^{o}$F and 86$^{o}$F) on October 9, 1998 and July 20, 1999, respectively, and the tensile strains were two to three times higher than when the HMA temperature was approximately 10$^{o}$C (50$^{o}$F) on February 4, 1999.

Replicating each sensor location provided the opportunity to look at the repeatability of the sensors. Figures F.15 through F.22 contain plots comparing the difference between the strains measured at the same spatial location within the panel but for different panels. Many of the sensor locations were cored before the UTW was reconstructed to determine the thickness of the HMA and the overlay at these locations and to determine the as-built depth of the sensor. A summary of this information is provided in table F.2. A photo log of the cores taken from all three test sections is provided in appendix G. The core information helps to provide insight into the cause of the difference between strain data measured at the same location and orientation but different panels. Unfortunately sufficient time was not available to core each sensor location. A summary of the strains measured at each location is provided below.

***Midpanel Adjacent to Inside Longitudinal Joint***

Strains measured along the inside longitudinal joint are lower than the strains measured at midpanel and along the L/S joint. This indicates the short panel size produces a smaller crack width capable of transferring load across to the adjacent panel allowing the group of panels to act as one continuous slab. Figure F.15 shows relatively good repeatability between the strains measured at this location in panels D and E. (See figure B.2 for the location of panel D and E and sensor locations.) Cores taken at these locations revealed the HMA was not bonded to the UTW and that some raveling had occurred on the bottom of the core pulled from panel E.

***L/S Longitudinal Joint***

The largest strain measured was along the L/S joint where there edge has less support than along the interior edges. Very little data was obtained at the bottom of the UTW at this location because the strain gages failed early. The strain measured in panel C was consistently higher than panel B. Coring each location revealed the HMA to be severely stripped. The HMA from the core in panel B had turned completely into unbound aggregate and approximately 75 percent of the HMA was stripped to the point of being unbound aggregate in the core from panel C. The combination of an edge loading and the HMA below the overlay being severely stripped and raveled resulted in high strains along the L/S joint.

***Corner***

Strains measured in the corner of the panel were also very high. A core pulled from panel C showed the HMA had delaminated between lifts and the bottom of the HMA had raveled. The HMA of the core taken from the corner of panel E was mostly raveled. The HMA was still in significantly better condition than the cores pulled along the L/S joint explaining why the strain measured in the corner tended to be lower than those along the L/S joint. Figure F.18 shows the strains measured at the corner in panel E and C to be relatively consistent with the exception of the strains measured at the bottom of the UTW in panel E being higher than those measured 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 compared to panel C. The strain gages placed in the corner were actually located 305 mm (12 in) away from the transverse and longitudinal edges. A core was pulled from panel E directly in the corner and the asphalt from the HMA was completely stripped resulting in unbound aggregate.

***Adjacent to Transverse Joint Near L/S Joint***

The strains measured in this location were relatively low compared to other locations. All strains measured at the bottom of the UTW were below 60 microstrain and below -40 microstrain at the top of the slab. Strains measured adjacent to the transverse joint near the L/S joint were relatively repeatable, as is shown in figure F.19. A core was pulled at this location from panel B. The asphalt was completely stripped at the bottom of the core. The repeatability of the strain measurements indicates similar conditions are present at this location in panel E.

***Adjacent to Transverse Joint in the Wheelpath***

Strain measured adjacent to the transverse joint in the wheelpath in panel D was consistently higher on the top sensor than in panel C because the sensor in panel D was approximate 10 mm (0.4 in) closer to the pavement surface. See table G.2. Approximately 30 percent of the strain is transferred across the joint between panels B and C and 40 percent between D and E. Cores taken from both locations revealed the UTW was bonded to the HMA. See figure G.14 and G.15.

***Midpanel***

Strains measured at midpanel typically ranged between 35 and 60 microstrain at the top of the UTW when the HMA temperature was approximately 10$^{o}$C (50$^{o}$F) or greater and strains measured at the bottom of the overlay ranged between 20 and 45 microstrain. The strains measured at midpanel in panel C were significantly higher than those measured in panel E. Cores were pulled from the midpanel of both panels E and C and the bond strength of the core from panel E was measured using the Iowa direct shear test (Iowa Test Method No. 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 form 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. The results from the strain measurements and the cores pulled from the test section indicate the HMA ravels at a faster rate along the joints where there is greater access for the water to enter the pavement structure compared to the center of the panel. The lane shoulder joint is the most difficult to keep sealed and therefore the HMA along this joint is more susceptible to stripping/raveling. The problem becomes magnified if the pavement is constructed with a gravel shoulder. Consideration should also be given to sealing the joints to limit the water 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, achieving the desired compaction during the original construction of the HMA pavement is more difficult 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 determining if 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 figures F.23 through F.29 along with the strains measured on U.S.-169. Dynamic strains measured on I-94 are significantly lower than those measured on U.S.-169 at all locations within each panel. The reduction in strain is a result of the increase in thickness and quality of the HMA on I-94 and an 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$^{o}$C (41$^{o}$F) resulting in compressive strains being generated at the bottom of the UTW when loaded. Increases in the temperature of the HMA also result in much smaller increases in strain on I-94 compared to U.S.-169. Significant increases in strain produced by the 40-kN (9,000-lb) FWD load begin to occur at HMA temperatures below 10$^{o}$C (50$^{o}$F) on U.S.-169 while significant increases in strain on I-94 are not seen until the temperature reaches 25$^{o}$C (77$^{o}$F).

The strains measured on I-94 were consistently lower than on U.S.-169 even when measurements were made at higher HMA temperatures. The exceptions to this are the strains at midpanel. Strains at midpanel on I-94 approach those measured at midpanel on U.S.-169. The UTW on I-94 is well bonded to the HMA similar to the bond conditions found when coring at midpanel on U.S.-169. In the regions along the edge of the panel where the UTW on U.S.-169 was found to be unbonded, the strains on U.S.-169 were significantly higher than on I-94. This indicates applying a load when the HMA temperature is high will produce similar strains in the UTW regardless of the thickness of the HMA layer even when a good bond is obtained.

# Chapter 5 Reconstruction of the Ultra-thin Whitetopping

The district decided to reconstruct the test sections in September 1999. The longitudinal joints were sawed to the depth of the overlay at the Jackson and School Street test sections on September 15 and then the concrete was removed with two IT28F front-end loaders on September 16. The longitudinal joints were sawed at the Main Street test section on September 16 and then the UTW was removed on September 17. Removing the concrete UTW took twice as long on the Main Street test section compared to the School and Jackson Street intersections because the severity of the distresses in this test section was not as high as the other two sections. A stronger bond between the HMA and UTW was also found at the Main Street test section and the HMA under the UTW was in better condition compared to the other two test sections. The Main Street test section also contained polyolefin fibers. The polyolefin fibers made it more difficult to separate the cracked slabs into smaller pieces.

All three test sections were paved with a roller screed on September 17 using a high early strength concrete mixture. The new pavement was 3.6-m (12-ft) wide and 178-mm (7-in) thick with 4.5-m (15-ft) joints. The joints contained 19-mm (0.75-in) epoxy-coated dowels. Blankets were used for curing. Contraction joints 3-mm (0.125-in) wide were sawed and sealed on September 18. The flexural strength of the concrete was 4 MPa (585 psi) at noon on September 19, 1999 when the sections were opened to traffic.

A distress survey was performed on the three sections in May 2002. See appendix H. All three sections exhibited a large number of transverse cracks. Seventy-one percent of the panels in the Jackson Street section were cracked and 76 and 82 percent of the panels were cracked in the School and Main Street sections, respectively. Some of the cracks are quite wide while the adjacent joints do not appear to be opening/closing. Many of these cracks could be the result of not sawing the joints in a timely fashion. The use of high early strength concrete increases the need to saw the joints early to prevent uncontrolled cracking.

## Cost Analysis

The cost of the UTW test sections at Jackson Street and School Street was $36.58 per yd$^2$. The cost to construct the UTW at Main Street was $38.10 per yd$^2$ because the cost of the polyolefin fibers cost more than the polypropylene fibers. The fibers for this project were donated but the cost estimates provided above represent costs accrued assuming the fibers were not donated and typical market value of the fibers. The UTW was reconstructed with a 178-mm (7-in) thick pavement costing $51.40 per yd$^2$. The costs of these sections were higher than that of a typical project because the size of the projects is small. It is difficult to make a comparison of the life-cycle costs between these two pavements alternatives because the UTW test sections were reconstructed prematurely as a result of the rough ride made worse by the blowpatch repairs and the cracking that developed in the 178-mm (7-in) thick pavement were most likely construction related.

## Conclusions

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

Distinct cracking patterns developed within each test section. The UTW test sections with a 1.2-m x 1.2-m (4-ft x 4-ft) joint pattern include corner breaks and transverse cracks. The corner breaks occurred primarily along the inside longitudinal joint and the lane/shoulder (L/S) longitudinal joint while the transverse cracks developed in the panels adjacent to the shoulder. The transverse cracks typically develop approximately 1/3 of the length of the panel away from the transverse joint. The Main Street test section with the 1.8-m x 1.8-m (6-ft x 6-ft) joint pattern performed significantly better than the Jackson and School Street intersection because the longitudinal joint does 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 UTW placed on thicker HMA pavements, such as on I-94.

The strains measured on I-94 were consistently lower than on U.S.-169 even when measurements were made at higher HMA temperatures. The reduction in strain is a result of the increase in thickness and quality of the HMA on I-94 and an increase in the bond strength between the two layers. Increases in the temperature of the HMA also produce much smaller increases in strain on I-94 compared to U.S.-169 except for the strains measured at midpanel. Strains at midpanel on I-94 approach those measured at midpanel on U.S.-169. It was found that applying a load when the HMA temperature is high produces similar strains in the UTW regardless of the thickness of the HMA layer when a good bond is obtained for the range of parameters included in this study.

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 the cores pulled from the test section indicate the HMA ravels at a faster rate along the joints where there is greater access for the water to enter the pavement structure. The lane shoulder joint is the most difficult to keep sealed and therefore the HMA along this joint was found to be more susceptible to stripping/raveling. Consideration should be given to sealing the joints to limit the water coming into contact with the HMA layer.

Achieving the desired compaction during the original construction of the HMA pavement is more difficult 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 determining if an UTW is an appropriate rehabilitation alternative.

# Chapter 6 Conclusions and Recommendations

Cores should be pulled from the pavement when evaluating whether UTW is a viable rehabilitation alternative 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 the asphalt layer is of uniform thickness and stripping/raveling has not occurred. If these conditions exist, UTW is a good option for rehabilitating asphalt pavements.