One-Year Performance Summary of Whitetopping Test Sections at the Mn/ROAD Test Facility

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ABSTRACT

A 345-mm (13.5-in) asphalt pavement on I-94, at the Minnesota Road Research (Mn/ROAD) test facility, was whitetopped with a fiber reinforced concrete overlay in October 1997. I-94 is a heavily trafficed roadway with approximately 1 million equivalent single axle loadings (ESALs) per year. Although this is not a typical application, it provided the opportunity to monitor the performance of the overlay under accelerated loading conditions and to evaluate ultra-thin whitetoppings as an overlay alternative for high volume roads. This test section was sub-divided into six test cells with various thicknesses, joint patterns and types of fibers. The test cells were instrumented with dynamic and static strain, temperature and moisture sensors. The purpose of this research project is to measure the static and dynamic response of the pavement under various applied and environmental loading conditions to optimize the design of ultra-thin whitetoppings.

Dynamic strain data was collected in conjunction with falling weight deflectometer testing and for known truckloads at various times of the year. Temperature, moisture and static strain data has been collected continuously since construction. The collected data has been used to determine the location and to quantify the magnitude of the maximum strain produced by both environmental and applied loads for various ultra-thin whitetopping designs. The strains can be used to predict the expected mode of failure and performance life of each test cell. Based on this, an optimum ultra-thin whitetopping design can be identified.

Subject Area: Full-Scale Test Tracks, whitetopping, bonded overlays

Introduction

Ultra-thin whitetopping (UTW) refers to placing a thin concrete overlay directly on top of an existing distressed asphalt pavement. To achieve satisfactory 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 stresses. Typical applications would include low-volume pavements where rutting, washboarding or shoving is present; such as airport aprons, general aviation taxiways, parking lots or intersections.

The Minnesota Department of Transportation constructed two ultra-thin whitetopping projects prior to this research project. The first was constructed on May 30, 1995. It is approximately 30 m (100 ft) long and was constructed on a 4-lane divided highway near North Mankato, MN. The original pavement consisted of 305 mm (12 in) of asphalt over 510 mm (20 in) of aggregate base and a loamy clay subgrade. The UTW was constructed by milling the outside lane of the northbound roadway to a depth of 75 mm (3 in) and placing 75 mm (3 in) of polyolefin fiber reinforced concrete. On June 5-6, 1995, an ultra-thin was constructed on the inside lane, adjacent to the previous section. The concrete used in the southernmost 30 m (100 ft) of the project contained polypropylene fibers while the concrete in the remaining 10-m (30-ft) portion did not contain fibers. Panels 1.8 m x 3 m (6 ft x 10 ft) and 1.8 m x 3.7 m (6 ft x 12 ft) were sawed in both lanes. This section of highway carries approximately 390,000 equivalent single axle loadings (ESALs) per year. Today, many corner cracks exist in both the plain and fiber-reinforced sections but most of the cracks are tight. The cracks in the fiber reinforced concrete tended to be tighter than the cracks in the plain concrete section (1)(2). This test section was rehabilitated in the fall of 1998 when severely cracked panels were removed, the exposed bituminous cleaned, and the concrete replaced. Two other areas that were approximately 40 cm^2 (16 in^2) were also removed and replaced.

In 1996, a second project was constructed on LorRay Drive in North Mankato, MN. The existing pavement consisted of 300 mm (11.5 in) of full-depth asphalt over 91cm (3 ft) of silty loam backfill. The bituminous surface was severely rutted due to heavy traffic loadings moving at slow speeds. Three whitetopping test sections were inlayed over the existing bituminous surface by milling the pavement to the depth of the concrete overlay. A 150-mm (6-in) overlay with 3 m x 3.6-m (10 ft x 12-ft) panels was constructed in the southbound lanes south of U.S. 14, a 115 mm (4.5 in) overlay with 1.5 m x 1.8-m (5 ft x 6-ft) panels was constructed on the northbound lanes south of U.S. 14, and a 75-mm (3-in) overlay with 1.5 m x 1.8-m (5 ft x 6-ft) panels was constructed on the northbound and southbound lanes north of U.S. 14. The concrete contained polyolefin fibers at a rate of 15 kg/m^3 (25 lbs/yd^3). Panels were sized to match the joints in the curb adjacent to the pavement. This project is currently in good condition. Only a few corner cracks are present and most of these cracks are still very tight. There has been one minor pavement failure in the 75 mm (3 in) section. It was subsequently discovered that the concrete thickness was only 25 mm (1 in) at this location.

Project Description

Based on the performance of the first two UTWs, it was determined that UTW had the potential of being a viable rehabilitation alternative. The purpose of this research project is to further evaluate how thin whitetopping and UTW will perform in Minnesota and to determine what design features are desirable to optimize the life of the pavement.

This study included two projects; one was constructed on US-169, in Elk River, MN and the other on I-94, at the Minnesota Road Research (Mn/ROAD) test facility. Both were constructed in the fall of 1997. The focus of this paper will be on the project constructed on I-94 at Mn/ROAD. A map with the location of this project is provided in figure 1. The original pavement was constructed in 1993 of 345-mm (13.5-in) full-depth asphalt over a silty-clay subgrade and located between the 5-year and 10-year design test cells at Mn/ROAD. The bituminous pavement was in good condition. Very few cracks were present and all were of low severity.

The Mn/ROAD test road is part of a heavily trafficed interstate with over 750,000 ESALs per year. Although this is not a typical application, it provided the opportunity to monitor the performance of the overlay under accelerated loading conditions. It also allowed the evaluation of both UTW and thin whitetoppings as a rehabilitation alternative for high volume roads.

The test section was divided up into six separate test cells with various thicknesses and joint patterns. These cells were instrumented with temperature, moisture and dynamic and static strain sensors. The sensor layout for the 1.5 m x 1.8-m (5 ft x 6-ft) panels is shown in figure 2 and 3. The same sensor layout was used for the other test sections but the dimensions were adjusted for the different panel sizes. A description of each test cell and the instrumentation installed is provided in table 1.

These heavily instrumented sections are used to measure both the static and dynamic response of the pavement under various applied and environmental loading conditions. This research project will provide a better understanding of how to more accurately model thin and ultra-thin whitetopping so that a more efficient design method and performance prediction model can be developed.

Mix Designs

Two different mix designs were used on this project; one with polyolefin fibers and one with polypropylene fibers. The polyolefin fiber mix contained 15 kg/m^3 (25 lbs/yd^3) of fibers and the polypropylene mix contained 1.8 kg/m^3 (3 lbs/yd^3) of fibers. A maximum water to cementitious ratio of 0.44 was specified for the polyolefin mix and 0.40 for the polypropylene mix. A water reducer was used in both mixes. All concrete was required to have an air content of 6.5 $\pm$ 1.5 percent and a 3-day flexural strength of 27 MPa (400 psi) so the overlays could be opened to traffic in three days. Mix material sources and aggregate gradations are provided in table 2 and 3, respectively.

Construction

October 3, 1997, the existing asphalt was milled in each test section to the depth of the concrete to be placed. Over the next 20 days, the test section was heavily instrumented. Paving took place on October 23 after all of the sensors had been installed. A central mix plant located approximately 20 minutes from the site was used during construction. The concrete was batched in 7.6-m^3 (10-yd^3) loads that were mixed for 5 minutes before the concrete was loaded into a truck. The concrete was agitated in the truck while being transported to the site. The mix designs used are provided in table 4. The weather was unseasonably cold [high of 7 ^{o}C (45 ^{o}F), low of 4 ^{o}C (39 ^{o}F)] so the mixing water was heated. The concrete plant had difficulty mixing 7.6-m^3 (10-yd^3) batches, so each truck was filled with two 3.8-m^3 (5-yd^3) batches. This resulted in slower paving compared to that achieved on the companion project in Elk River, MN.

A 7.3-m (24-ft) wide slip-form paver was used. Paving was performed off a double string line so that the final elevation of the concrete could be maintained more accurately for improved ride quality and depth control. The pavement was textured using longitudinal astro-turf drag and random 19-mm (3/4-in) transverse tining. In general, a smooth finish was obtained using standard finishing techniques. There were some areas in the polyolefin concrete that had clumps of fibers lying on the finished pavement surface that increased the roughness of the finish. These clumps eventually broke apart under traffic leaving small imperfections in the surface.

The overlay was covered with wet burlene approximately one hour after the final portion of the concrete was placed. The contractor began sawing transverse joints at 7:00 AM the following morning. The sawing was delayed because of the cold weather. Immediately after the joints were sawed, the sawing residue was washed off the surface and the concrete was covered with wet burlene and cold weather blankets. Cells 96, 97 and 97b were washed off and covered with 2 layers of cloth lined plastic and cold weather blankets because there was not a sufficient amount of burlene available for the entire project. The overlay was uncovered and the longitudinal joints were sawed on October 27. On the following day, the joints were sealed with hot poured rubber sealant and linseed oil was applied to the pavement surface at a rate of 3.7 m^2/liter (17 yd^2/gal). The whitetopped section was opened to traffic on October 30, 1997.

The California profilograph was used to measure ride quality. Using a 2.5 mm (2/10in) blanking band, a surface profile of 0 mm/km (0 in/mile) was measured on the outside lane and 2 mm/km (0.15 in/mile) on the inside lane. The ride quality bonus for this project was twice that typically specified by Mn/DOT Specification 2301.3Plc (revised 10.23.96) and 100 percent of the pavement placed, including that adjacent to the headers, received full bonus.

Hardened Concrete Properties

Numerous beams and cylinders were cast for material property testing. On the day after paving, the beams and cylinders were stripped from their molds and all but seven beams were placed in an environmental chamber and cured according to ASTM C31. The seven remaining beams were left along the side of the pavement and the same curing techniques used on the whitetopping were applied to these seven beams. A summary of the comparisons between the polypropylene concrete and the polyolefin concrete material properties is provided below.

The compressive strengths are given in table 5. The compressive strengths for the polypropylene concrete were consistently higher than the polyolefin concrete for all five specimen ages. There are over 8 times as many fibers in the polyolefin mix than in the polypropylene mix. Replacing this much concrete in the mix, with fiber, can be a contributing factor to the lower compressive strengths. Some of this decrease in strength may also be attributed to a lower w/c ratio (0.38 vs. 0.41). Even though the compressive strengths for the polyolefin mix were lower, both mixes met Mn/DOT specifications. The increased fiber content and higher w/c ratio also resulted in a slightly lower elastic modulus for the polyolefin mix. This is shown in table 6. The Poisson's ratio for both mixes was 0.19, as seen in table 6. It should also be noted that the elastic moduli and Poisson's ratio measured for the fiber reinforced concrete is comparable to that of conventional concrete.

Table 7 shows flexural strengths for beams cured according to ASTM C31. The two mixes had comparable flexural strengths. Although the fiber has little strength in compression, it is much stronger in tension. Therefore, the increase in fiber content for the poylolefin mix does not significantly reduce the flexural strength, as it did with the compressive strength. The slight reduction in the flexural strength of the polyolefin mix is most likely due predominately to the increase in w/c ratio. Both mixes have more than sufficient strength to meet Mn/DOT's strength requirements.

The flexural strengths for the specimens cured according to ASTM C31 are over 25 percent greater than the strengths for the specimens cured in the field. The flexural strengths for the specimens cured in the field are provided in table 7. It is not clear why the earlier strengths for the polyolefin concrete are so much lower than the early strengths for the polypropylene concrete for the field cured specimens. A possible explanation for this may be because the wind had blown some of the blankets back on this portion of the whitetopping; thereby exposing the overlay and possibly some of the polyolefin concrete beams to wind and the cold air.

The extensive material property testing on the I-94 project revealed that the fiber reinforced concrete for these mix designs and conventional concrete have similar material property characteristics. The polypropylene concrete typically showed a slightly higher compressive strength and elastic modulus compared to the polyolefin concrete, most likely due to decreased fiber content and a lower w/c ratio. The flexural strengths for the two mixes were comparable. The influence of curing on concrete strength was also emphasized. The specimens continuously moist cured in an environmental room exhibited significantly higher strengths than those cured in the field. The concrete overlay retained heat and moisture more efficiently than the field cured test specimens but not as well as the specimens cured in a laboratory environment, especially given the cold field conditions. Therefore, the strength of the concrete overlay most likely falls somewhere between the strength of the field-cured beams and the strength of the beams cured according to ASTM C31.

Material Characterization of Existing Asphalt

The optimum 120/150 penetration binder content for the existing asphalt concrete was selected using the 75-blow Marshall mix design method. The same mix design was used to construct a similar test section at the Mn/ROAD test facility. In that test section, the pavement was cored immediately after field compaction, prior to any traffic loadings. Resilient modulus (ASTM D4123) temperature susceptibility testing was performed on the separated lifts of the field cores. The resilient modulus measured for each layer can be considered representative of the resilient modulus of the corresponding layer in the asphalt pavement the whitetopping was constructed over. The moduli measured in the companion section below 75 mm (3 in) from the existing pavement surface were used in this study (4).

Economic Analysis

An economic analysis was performed to determine the economical feasibility of whitetopping. The analysis included the estimated cost of the test sections constructed at Mn/ROAD and estimates of what more typical whitetopping projects would cost since there were several factors that increased the cost of the test sections above what would normally be anticipated. For example, the project letting was in June and awarded in July, several months after the majority of the concrete paving projects for 1997 had been awarded. This late letting/award most likely increased the prices quoted compared to what would have been observed with an earlier letting because of the lack of flexibility available to the bidding contractors in scheduling the construction around their larger paving projects. The project also required the use of a slip-form paver which has a significantly higher mobilization cost when compared to other paving equipment, such as the clarey screed. Requiring a slip-form paver also limited the number of bidders on the project to two.

The cost per square meter of each test section is listed in table 8 along with estimated costs for more typical small projects (10,000 - 30,000 m^2) and typical larger projects (> 30,000 m^2). The estimated costs include the following assumptions and associated expenses:

1. The costs of mobilization, striping and joint sealing are prorated to a unit cost per square meter.

2. A large project is of sufficient size to justify the time and cost required for a contractor to bring in his own mobile concrete plant. This would reduce the concrete cost from \$125 per m^3 (cost of concrete when purchased from a ready-mix company) to \$90 per m^3. A larger project would also significantly reduce the mobilization price per square meter.
3. The polyolefin fibers were donated for the Mn/ROAD project. When estimating expected costs, the polyolefin concrete cost was assumed to be \$60.00 per m^3 more than the concrete containing polypropylene fibers.
4. Joint sealing was estimated to cost \$0.80 per lineal meter based on a statewide average price from 1997 for sealing a 25 mm deep and 6 mm wide single saw cut. Joint sealing was performed under a supplemental agreement and was not included as a bid item for the Mn/Road whitetopping.

Several observations can be made based on the estimates provided in table 8. Comparing the cost of Cell 93 and 95 shows the additional cost of the polyolefin fibers is somewhat offset by the savings incurred when using larger panel sizes (1.2 m x 1.2 m vs. 1.5 m x 1.8 m) because the larger panels require less joint sawing and sealing. The reduction in cost when using larger panels is also seen when comparing the cost of Cell 96, 97 and 97b. The difference between the cost of the 3 m x 3.7-m (10 ft x 12-ft) panels in Cell 97b and the 1.5 m x 1.8-m (5 ft x 6-ft) panels in Cell 96 is almost large enough to offset the additional cost of dowels in Cell 97b. The significant cost variations between the overlays increases the importance of looking at both the cost of each design variable and the effectiveness of that variable on increasing the serviceability life of the overlay.

One-Year Performance Summary

Each test cell has been monitored regularly. Dynamic strain data was collected in conjunction with falling weight deflectometer (FWD) testing at six different times during the first year. The FWD data was used to monitor the stiffness of the structure at different locations of the panel throughout the year. Distress surveys were also performed. Data collection dates are given in table 9 and a summary of the test results is provided below.

FWD Testing

The key to long-term performance of UTWs is to maintain a good bond between the overlay and the existing asphalt pavement so that the two layers act as one monolithic section. In a fully bonded condition, the neutral axis can drop down in the asphalt layer so the concrete overlay is completely in compression. As the overlay begins to debond, tensile stress will develop at the bottom of the overlay. These stresses will increase as the strength of the bond decreases so the life of the overlay will shorten (3). See figure 4. The stiffer the asphalt layer is relative to the concrete layer the further down the neutral axis will drop. The use of short joint spacing helps reduce bending stresses in UTWs because curling and warping stress are reduced and the slab will tend to *deflect* downward as oppose to bending under applied wheel loads (3). See figure 5.

Knowing the strains at both the top and bottom of the slab makes it possible to estimate the degree of bonding between the asphalt and concrete. The magnitude of the strains also provides an indication of the expected life of the overlay. FWD testing was performed directly above each dynamic strain sensor in each test cell. These sensors are located at six different areas within the panel. Each location has one sensor located at the bottom of the overlay and one sensor 25 mm (1 in) from the top of the overlay. See figure 3. FWD testing was performed at 3 different load magnitudes [27 kN (6 kips), 40 kN (9 kips) and 67 kN (15 kips)] with three drops per load. The strain analysis below is based on the average strain for the three 40-kN (9-kip) loads.

All strains measured at the bottom of the concrete slab between November 1997 and March 1998 in the 75-mm (3-in) test sections were below +10 microstrain (tension is positive). Most strains were in compression. This indicates that there is a good bond between the two layers. Strains at the bottom of the slab for the 100-mm (4-in) test section, measured within this same time period, were very low but slightly higher than the strains measured in the 75-mm (3-in) test sections. All of the strains measured at the bottom of the overlay in the 150-mm (6-in) test sections were in tension. Most of the strains measured at the top of the slab were greater than those measured at the bottom of the slab indicating that the neutral axis did shift down below mid-depth of the concrete layer. The increased stiffness do to the additional thickness of the overlay prevented the neutral axis from shifting down further as was seen in the 100-mm (4-in), and even more so in the 75-mm (3-in) overlays. All strains measured at the bottom of the slab between November 1997 and March 1998 in the 150-mm (6-in) sections were less than +20 microstrain. The strain measurements indicate that by increasing the concrete thickness the strains at the bottom of the concrete can increase. This is because the flexural rigidity of the overlay increases causing the neutral axis to shift upward. Even though increasing the thickness of the concrete will increase the load carrying capacity of the overlay it also shifts the neutral axis upward forcing a larger portion of the load to be carried by the overlay and less by the asphalt. Therefore, increasing the thickness of the overlay may not always increase the serviceability life. Similar findings were found in a study performed by Mack et al. (5) using finite element analysis.

Strains measured at the bottom of the slab were plotted against accumulated ESALs for each test location. Accumulated ESALs were calculated for all test sections assuming a 150-mm (6-in) portland cement concrete slab and a terminal serviceability of 2.5. Figure 6 shows strains measured at midpanel adjacent to the lane/shoulder joint for the test section with 75 mm (3 in) thick 1.2 m x 1.2-m (4 ft x 4-ft) panels. Figure 6 indicates that there is not a strong relationship between increasing strains and accumulated ESALs at this stage in the life of the overlay. A stronger relationship was found between the stiffness of the asphalt and the measured concrete strains. See figure 7. As temperatures increased and the resilient modulus of the asphalt decreased, concrete strains increased. Increases in the resilient modulus of the asphalt increases the flexural rigidity of asphalt layer relative to that of the concrete overlay causing the neutral axis to shift downward and strains at the bottom of the concrete to decrease. The same trend was present at all test locations for the 75-mm (3-in) and 100-mm (4-in) overlays. This trend appeared to be stronger for the test section with 75 mm (3 in) thick 1.2 m x 1.2-m (3 in-4 ft x 4-ft) panels when compared to the thicker overlays. One reason is because it is more difficult to overcome the increased flexural rigidity of a thicker concrete overlay by increasing the stiffness of the asphalt since the thickness of a layer has a cubed affect on the flexural rigidity unlike the elastic modulus. See equation 1 (6). Also, the panel deflects downward more easily under an applied load when overlay is thin and the panels are small. The magnitude of the strains measured in 150-mm (6-in) overlays tended to be dependent on the number of ESALs accumulated as well as the stiffness of the asphalt. See figure 8 and 9.

$$D = \frac{Eh^3}{12\left(1-\mu^2\right)} \qquad \text{Equation 1. (6)}$$

D = Flexural rigidity
E = Modulus of elasticity
h = Thickness of layer
μ = Possion's ratio

All of the strains measured are still relatively low after one year and 750,000 ESALs. A more in-depth comparison between test sections can be made after more vehicle loads are applied and the sections begin to deteriorate.

Distress Survey

Distress surveys were performed on the test sections at six different times during the first year following construction. The first cracks were found in June of 1997. Table 10 summarizes the results of the distress surveys. At this time, no cracks are present in the 150-mm (6-in) overlays. Most of the corner breaks in the other test cells occurred between March and June of 1997. This could be the result of high negative temperature gradients [0.7 C/cm (-3.3 F/in)] that were measured in May 1997. Corners of the slab curl upward in the presence of a negative gradient, thereby reducing the support in these areas. This increases the potential for cracking when these partially supported areas are loaded. Most of the transverse cracks were found in January 1999. All of the cracks found during this time were reflection cracks that originated in the existing asphalt and reflected up into the overlay. This cracking most likely occurred when pavement temperatures dropped below –25 °C (-13 °F) during the first week of January 1999.

After 15 months, very little cracking has occurred and all are low severity cracks. The 75-mm (3-in) test section with 1.2 m x 1.2-m (4 ft x 4-ft) panels has the largest amount of cracking. It is too early to determine if the increase in cracking is significant since very little cracking has occurred in any of the sections. Over 70 percent of the cracking is grouped in regions above transverse cracks in the asphalt. This stresses the importance of not placing bonded overlays on asphalt pavements that have an excessive amount of deteriorated transverse cracks.

Summary

Six whitetopping test sections were constructed at the Mn/ROAD test facility. Fiber reinforced concrete was used to construct the whitetopping sections. Two different types of fibers were investigated, polyolefin and polypropylene fibers. Both types of fiber produced concrete having material properties comparable to that of conventional concrete. These test sections were instrumented with temperature, moisture and static and dynamic strain sensors. FWD testing and distress surveys were performed throughout the year. Results from the dynamic strain measurements indicate that a good bond was obtained between the asphalt and concrete layers. It was also observed that the overlay thickness and the asphalt stiffness significantly affect the strains in the overlay because of their influence on the location of the neutral axis. For this reason, increasing the thickness of the overlay will not always result in lower strains. Almost all of the measured strains were very low and there are no cracks present in the 150-mm (6-in) test sections and very little cracking in the 100 and 75-mm (4 and 3-in) test sections after 15 months of service and 750,000 ESALs. It is still too early to identify an optimum overlay design.

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DISCLAIMER

TABLE 1. Summary of Mn/ROAD whitetopping test sections.

Test Cell Description	***Sensor***	***No. of Sensors***
Cell 93: 100mm – 1.2 m x 1.2-m Panels (4 in – 4 ft x 4-ft) Polypropylene Fibers	Dynamic Strain Dynamic Asphalt Foil Strain Gage Static Strain Thermocouple Moisture	32 2 8 14 12
Cell 94: 75 mm – 1.2 m x 1.2-m Panels (3 in – 4 ft x 4-ft) Polypropylene Fibers	Dynamic Strain	32
Cell 95: 75mm – 1.5 m x 1.8 m Panels (3 in – 5 ft x 6 ft) Polyolefin Fibers	Dynamic Strain Dynamic Asphalt Foil Strain Gage Static Strain Thermocouple Moisture	32 2 8 12 12
Cell 96: 150 mm – 1.5 m x 1.8-m Panels (6 in – 5 ft x 6-ft) Polypropylene Fibers	Dynamic Strain	32
Cell 97: 150 mm – 3 m x 3.7-m Panels (6 in – 10 ft x 12-ft) Polypropylene Fibers	Dynamic Strain Dynamic Asphalt Foil Strain Gage Static Strain Thermocouple Moisture	32 2 8 16 12
Cell 97b: 150 mm – 3 m x 3.7-m Panels (6 in – 10 ft x 12-ft) Polypropylene Fibers 25mm x 381mm Dowels	-None-	–
	Total	**268**

TABLE 2. Concrete material sources for Mn/ROAD overlays.

Cement	Holnam- Mason City, IA
Class C Fly Ash	None
Fine Aggregate	Barton-Osseo; Pit No. 127003
Coarse Aggregate (19 mm minus)	Barton-Osseo; Pit No. 127003
Coarse Aggregate (10 mm minus)	Barton-Osseo; Pit No. 127003
Polypropylene Fibers	*Fibermesh*
Polyolefin Fibers	*3M*
Air Entrainer	*Conchem-Conchem Air*
Water Reducer	*Conchem-Polyheed-N*

TABLE 3. Concrete coarse and fine aggregate gradations for Mn/ROAD overlays.

COURSE AGGREGATE			**FINE AGGREGATE**	
	Percent Passing			***Percent Passing***
Sieve Size	***CA-50***	***CA-80***	***Sieve Size***	***FA***
51 mm (2 in)			**9.5 mm (0.375 in)**	100
38 mm (1.5 in)			**4.75 mm (No. 4)**	100
32 mm (1.25 in)			**2.36 mm (No. 8)**	92
25 mm (1 in)	100		**1.18 mm (No. 16)**	72
19 mm (0.75 in)	97		**600 µm (No. 30)**	48
16 mm (0.625 in)			**300 µm (No. 50)**	19
13 mm (0.5 in)			**150 µm (No. 100)**	4
9.5 mm (0.375 in)	44	100	**75 µm (No. 200)**	
4.75 mm (No. 4)	5	60	**F.M.**	2.65

TABLE 4. As-placed concrete mix proportions, at the Mn/ROAD research site.

	Concrete with Polypropylene Fibers	***Concrete with Polyolefin Fibers***
Water/Cementitious Ratio	0.38	0.41
Cement, kg/m^3 (lbs/yd^3)	386 (650)	386 (650)
Class C Fly Ash, kg/m^3 (lbs/yd^3)	0	0
Fine Aggregate, kg/m^3 (lbs/yd^3)	704 (1187)	764 (1287)
CA (19 mm minus), kg/m^3 (lbs/yd^3)	949 (1600)	890 (1500)
CA (10 mm minus), kg/m^3 (lbs/yd^3)	164 (277)	164 (277)
Fiber Content, kg/m^3 (lbs/yd^3)	2 (3)	15 (25)
Measured Air, %	5.75	7.2
Measured Slump, mm (in.)	65 (2.5)	50 (2)

TABLE 5. Compressive strengths for specimens cured according to ASTM C31.

	Concrete with Polypropylene Fibers *MPa (psi)*	**Concrete with Polyolefin Fibers** *MPa (psi)*
1-Day	13.8 (2000)	11.0 (1600)
3-Day	26.9 (3900)	20.0(2900)
7-Day	33.1 (4800)	29.0 (4200)
14-Day	37.9 (5500)	33.1 (4800)
28-Day	42.1 (6100)	36.5 (5300)

TABLE 6. Elastic modulus and Poisson's ratio for specimens cured according to ASTM C31.

	ELASTIC MODULUS		**POISSON'S RATIO**	
	Concrete with Polypropylene Fibers *MPa (psi)*	**Concrete with Polyolefin Fibers** *MPa (psi)*	**Concrete with Polypropylene Fibers**	**Concrete with Polyolefin Fibers**
7-Day	31,026 (4.5 x 10^6)	29,647 (4.3 x 10^6)	0.19	0.19
28-Day	33,094 (4.8 x 10^6)	30,337 (4.4 x 10^6)	0.19	0.19

TABLE 7. Flexural strengths for specimens cured in the field and according to ASTM C31.

	CURED ACCORDING TO ASTM C31		**CURED IN THE FIELD**	
	Concrete with Polypropylene Fibers *MPa (psi)*	**Concrete with Polyolefin Fibers** *MPa (psi)*	**Concrete with Polypropylene Fibers** *MPa (psi)*	**Concrete with Polyolefin Fibers** *MPa (psi)*
4-Day	5.1 (750)	4.6 (670)	3.9 (570)	3.0 (440)
7-Day	5.9 (850)	5.8 (840)	3.6 (520)	3.2 (460)
28-Day			4.3 (620)	4.3 (630)

TABLE 8. Cost summary of overlay designs constructed at the Mn/ROAD site.

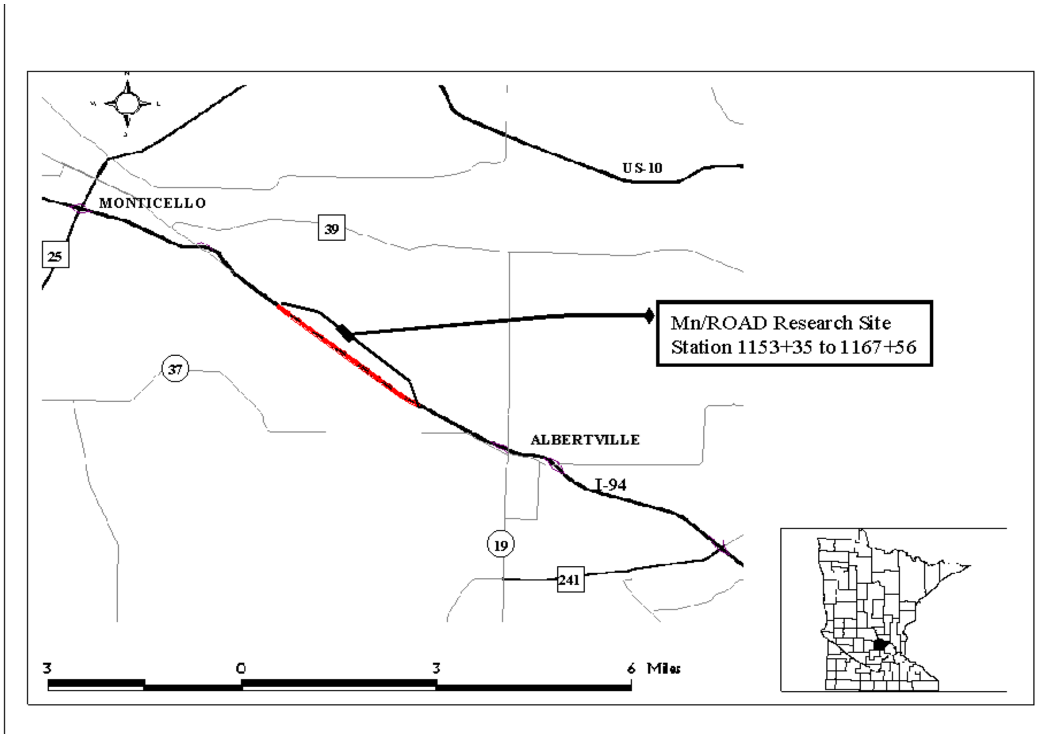
Overlay Design	***Cost of Mn/ROAD UTW Test Sections (per m²)***	***Estimated Cost (Small Project: 10,000-30,000 m²) (per m²)***	***Estimated Cost (Large Project: >30,000 m²) (per m²)***
Cell 93: 100mm – 1.2 m x 1.2 m Panels (4 in – 4 ft x 4 ft) Polypropylene Fibers	\$42.13	\$30.90	\$24.90
Cell 94: 75 mm – 1.2 m x 1.2 m Panels (3 in – 4 ft x 4 ft) Polypropylene Fibers	\$36.58	\$27.90	\$21.90
Cell 95: 75mm – 1.5 m x 1.8 m Panels (3 in – 5 ft x 6 ft) Polyolefin Fibers	\$38.66	\$31.08	\$24.78
Cell 96: 150 mm – 1.5 m x 1.8 m Panels (6 in – 5 ft x 6 ft) Polypropylene Fibers	\$48.00	\$33.78	\$29.28
Cell 97: 150 mm – 3 m x 3.7 m Panels (6 in – 10 ft x 12 ft) Polypropylene Fibers	\$46.36	\$31.50	\$26.40
Cell 97b: 150 mm – 3 m x 3.7 m Panels (6 in – 10 ft x 12 ft) Polypropylene Fibers Dowels (381mm long; dia.=25mm)	\$49.36	\$34.80	\$29.70

TABLE 9. Data collection dates for the first year following construction.

Type of Data Collection	***Data Collection Period***					
	1	***2***	***3***	***4***	***5***	***6***
FWD and Strain Data	Nov. ‘97	Dec. ‘97	Feb. ‘98	Mar. ‘98	June ‘98	Oct. ‘98
Distress Survey	Nov. ‘97	Feb. ‘97	Mar. ‘98	June ‘98	Aug. ‘98	Oct. ‘98

TABLE 10. Summary of distress surveys.

Month	***No. of Panels Cracked***	***No. of Corner Breaks***	***No. of Transverse Cracks***
CELL 93			
Feb. 1998	0	0	0
June 1998	6	4	3
Oct. 1998	1	1	0
Nov. 1998	2	1	1
Jan. 1999	1	0	2
Total (%)	***2.2 %***	***1.3 %***	***1.3 %***
CELL 94			
Feb. 1998	0	0	0
June 1998	14	13	4
Oct. 1998	4	4	0
Nov. 1998	2	1	0
Jan. 1999	6	1	6
Total (%)	***5.7 %***	***4.2 %***	***2.2 %***
CELL 95			
Feb. 1998	0	0	0
June 1998	3	3	0
Oct. 1998	0	0	0
Nov. 1998	0	0	0
Jan. 1999	1	0	1
Total (%)	***1.6 %***	***1.3 %***	***0.4 %***



(1 mile = 1.61 kilometers)

FIGURE 1. Location of Mn/ROAD test facility.

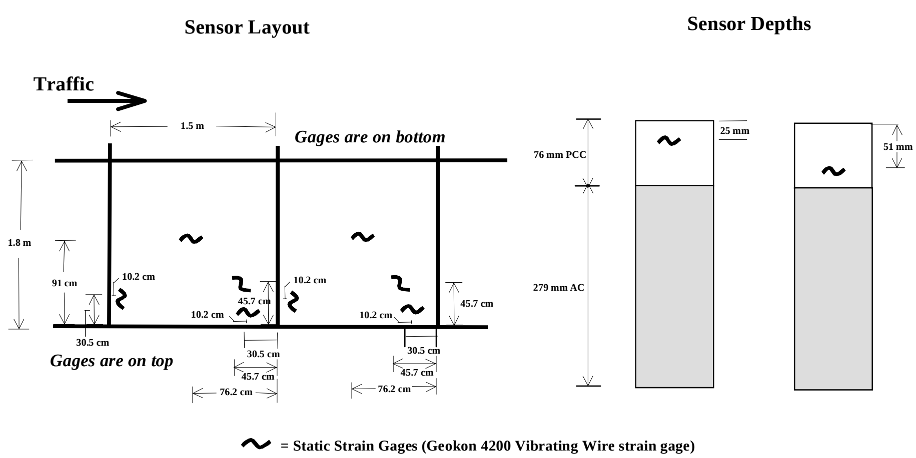
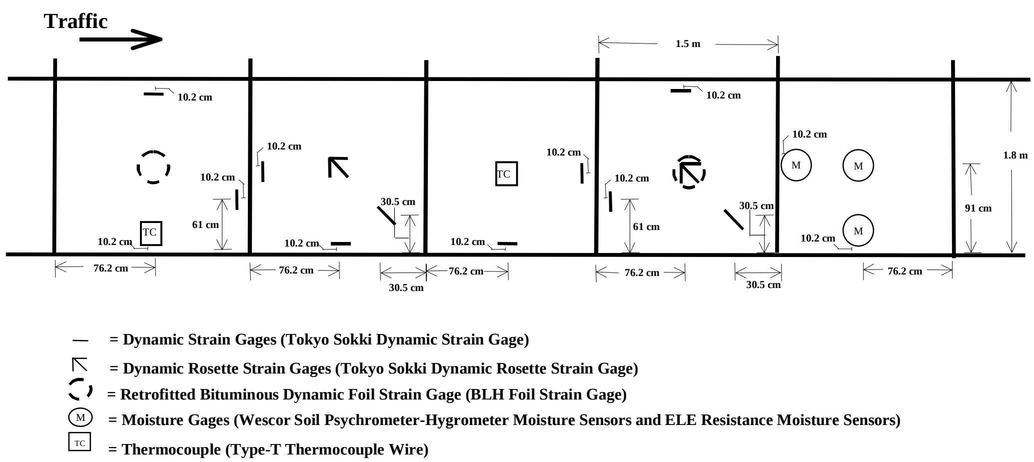


FIGURE 2. Typical static strain sensor layout for 1.5 m x 1.8-m (5 ft x 6-ft) panels.

Typical Sensor Layout for Climatic and Dynamic Strain Sensors



Typical Depths for Climatic and Dynamic Strain Sensors

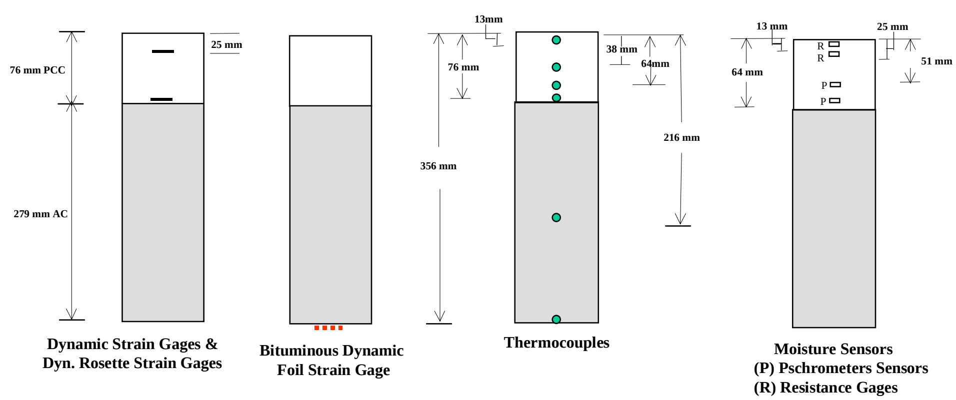


FIGURE 3. Typical climatic and dynamic strain sensor layout for 1.5 m x 1.8-m (5 ft x 6-ft) panels.

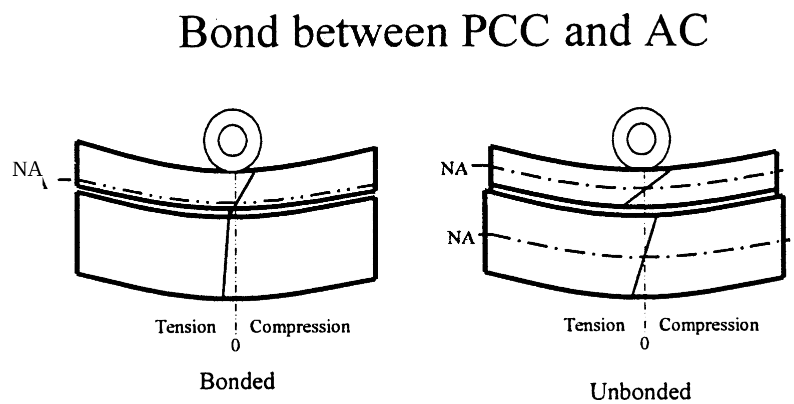


FIGURE 4. Effect of bonding on the performance of ultra-thin whitetoppings (3).

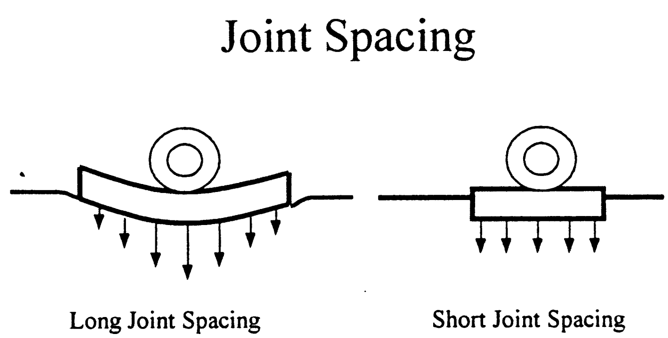


FIGURE 5. Effect of joint spacing on the performance of ultra-thin whitetoppings (3).

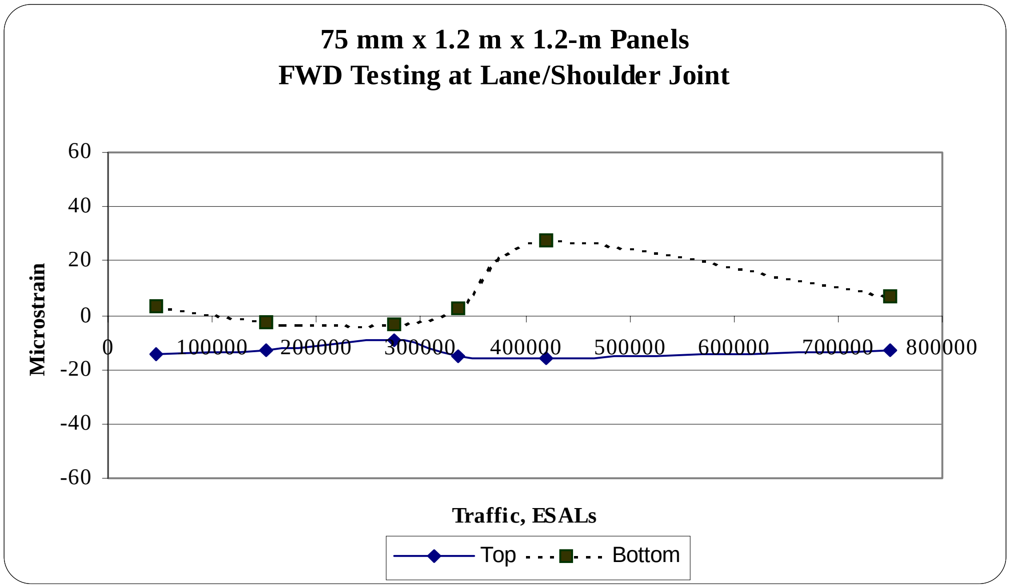


FIGURE 6. Relationship between strain and accumulated ESALs for Cell 94.

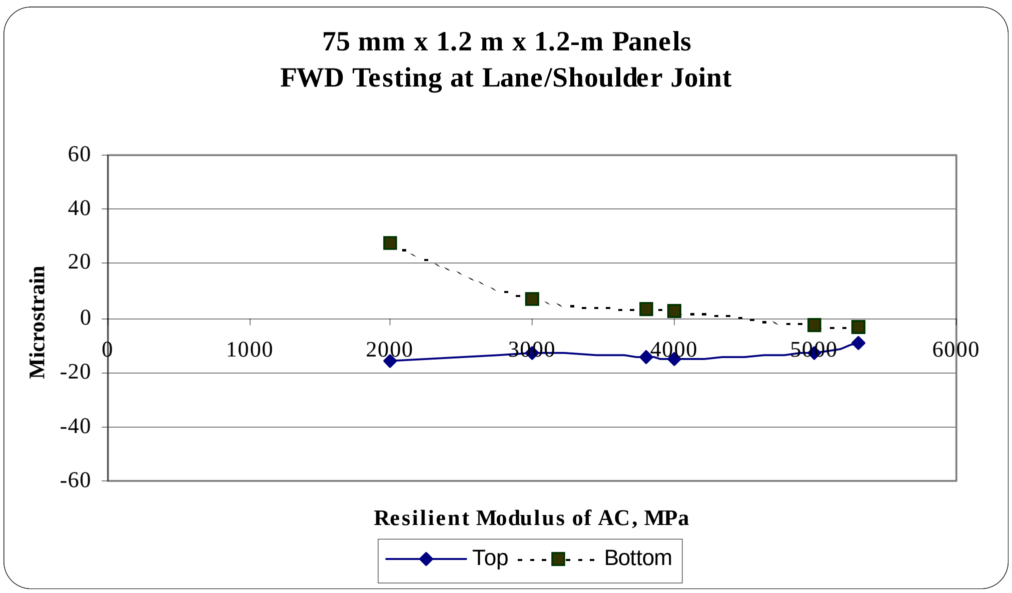


FIGURE 7. Relationship between strain and resilient modulus of AC for Cell 94.

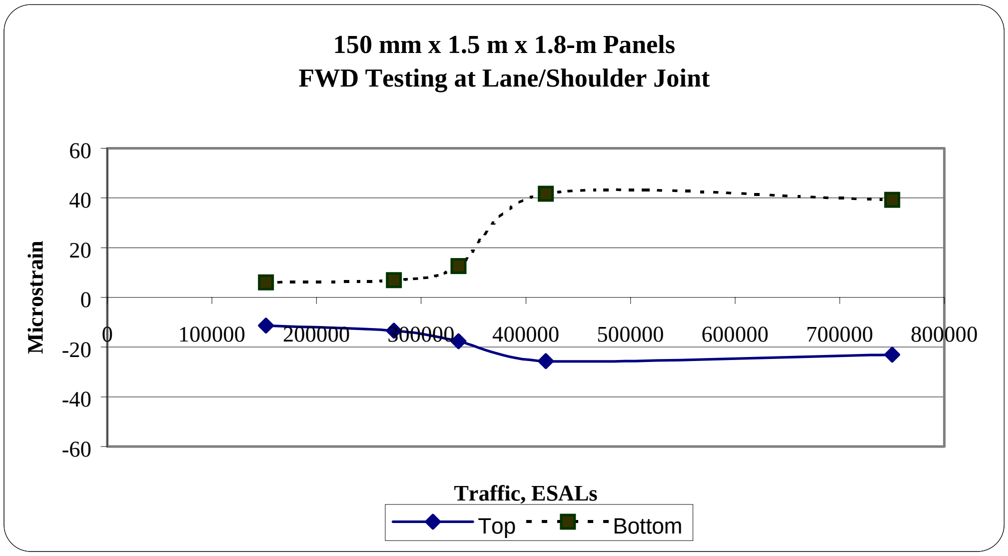


FIGURE 8. Relationship between strain and accumulated ESALs for Cell 96.

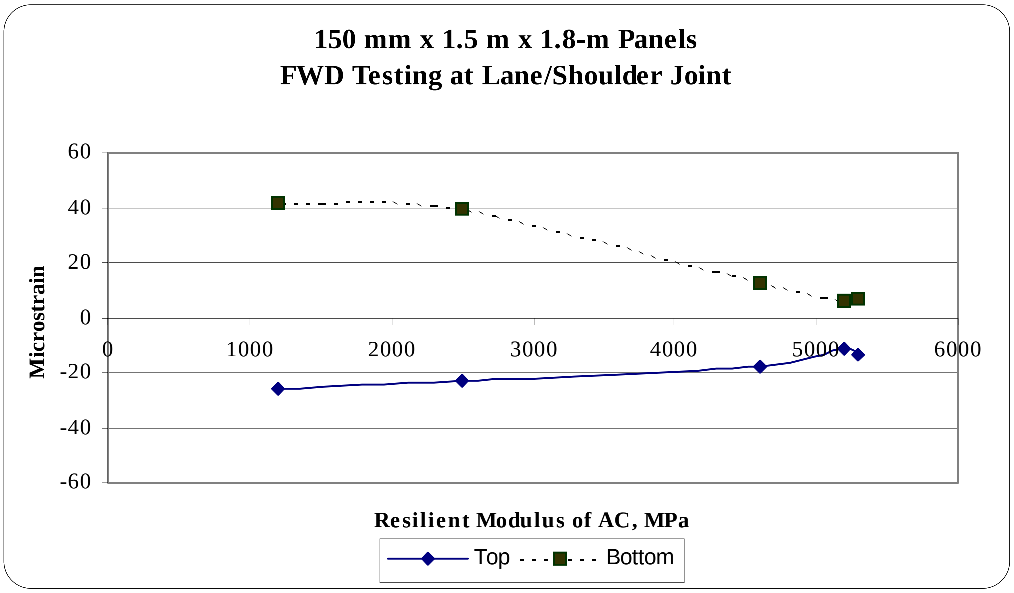


FIGURE 9. Relationship between strain and resilient modulus of AC for Cell 96.