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REHABILITATION OF ELLAVILLE WEIGH STATION WITH ULTRA-THIN WHITETOPPING

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ABSTRACT

In 1996, the Florida Department of Transportation (FDOT) began a research project to evaluate design, construction and performance of the Ultra-Thin Whitetopping (UTW) on asphalt pavements. Three UTW test tracks were constructed at the FDOT materials and research facility in Gainesville, Florida. Test results showed excellent bond between the UTW and asphalt surface. The Falling Weight Deflectometer tests indicated a significant improvement in the load carrying capacity of the concrete/asphalt composite pavement. The success of this research prompted the FDOT to construct the first UTW project at the Ellaville Truck Weigh Station on I-10 in Northwest Florida. This rehabilitation project included the placement of UTW on the existing asphalt pavement which had experienced severe rutting and cracking problems. Layer thickness for the UTW were 80 mm and 100 mm. The joint spacing for the UTW panels were 1.2 m and 1.6 m. High early strength concrete was used in this project. Polypropylene fibers were included in the concrete for the sections on the west side of the weight platform and plain concrete was used on the east sections. The joints on the east section were sealed with silicone sealant while the joints in the west side section were left unsealed. This paper discusses the implementation of the UTW, which includes design, specification, construction and performance at the Ellaville Weigh Station.

Keywords: ultra-thin whitetopping; implementation; case study; design; rehabilitation; joint sealing, high early strength concrete, fibers, Falling Weight Deflectometer; service life.

INTRODUCTION

For over twenty years, concrete overlay on asphalt pavements, commonly known as whitetopping, has been a viable alternative for rehabilitating deteriorated asphalt pavements and upgrading their structural capacity. The thickness of conventional whitetopping has been at least 125 mm. In Florida, a whitetopping project was constructed on US 1 in 1987. Thicknesses for the concrete overlay included 150 mm, 175 mm, and 200 mm. The project has shown good performance and has developed very low faulting and cracking.

For the past seven years, many state highway agencies, counties and cities have placed thinner concrete overlays than the conventional whitetopping (1). These concrete overlays have been commonly referred to as Ultra-Thin Whitetopping (UTW). They have ranged in thickness from 50 to 100 mm. Following the encouraging results from the first experimental project in Louisville, Kentucky, over 70 UTW projects have been implemented in 18 states. Most were placed on deteriorated asphalt pavements in city streets, intersections, truck weigh stations and low to medium volume roads (2).

Some general guidelines for design and construction of UTW have been developed based on experience from individual projects (3). On a typical UTW project, the rutted asphalt pavement is likely to be milled to remove the rut and then resurfaced with a concrete overlay. Thicknesses for the UTW range from 50 to 100 mm and joint spacings range from 0.61 m to 1.83 m. The recommended minimum thickness of the asphalt layer beneath the UTW has been 50mm to 75 mm (3). It has been assumed that leaving 50 to 75 mm of the asphalt in-place after milling prevents deterioration and structural damage to the base layer, and insures a good bond between the asphalt and the UTW.

Based on surveys compiled by the American Concrete Pavement Association, early indications from the pavement condition surveys suggest good performance for most UTW projects. Design parameters such as UTW layer thickness, joint spacing, asphalt thickness, base support condition, and bonding to the asphalt surface have significant impact on the performance and longevity of the UTW. Another important issue is how to best support the panels at the outer edges of the pavement. Studies are underway to develop analytical and design models for UTW (4). However, these procedures are yet to be validated by field conditions.

Other important factors which require careful consideration are materials, concrete mixtures and construction techniques. Specific issues include the use of fibers in concrete, fast track mixture design, early saw cutting of freshly placed concrete and whether or not to use joint sealants.

In 1996, the Florida Department of Transportation (FDOT) started a research project to evaluate design, materials, construction and performance of the UTW. The ultimate goal was to implement this technique as a rehabilitation alternative for asphalt pavements. The research project involved the construction, testing and structural evaluation of three UTW test tracks in Gainesville (5). The success of this research project prompted the FDOT to implement the first UTW project to rehabilitate a severely rutted asphalt pavement at the Ellaville Truck Weigh-Station on Interstate 10 (I-10) in North Florida. This paper describes the design, construction, and evaluation of the Ellaville Weigh Station.

GAINESVILLE TEST TRACKS

Three UTW test tracks were constructed on a vacated maintenance yard behind the State Materials Office in Gainesville (5). The thickness of these tracks were 50 mm, 80 mm and 100 mm. Panel joint spacings were 0.91 m x 0.91 m, 1.22 m x 1.22 m, and 1.83 m x 1.83 m. The UTW was placed on original and milled asphalt surfaces. The asphalt beneath the three tracks varied from 25 mm to 50 mm. The test tracks were constructed using concrete mixtures with fibers except on short controlled sections in which were plain concrete was used.

The test tracks were loaded for more than eighteen months using trucks carrying concrete blocks. Over 60,000 (80 kN) ESALs were applied on the tracks. Tests using the Falling Weight Deflectometer (FWD) were performed on the original asphalt pavement and on the surface of the UTW prior and during the truck loading period. The FWD tests indicated significant improvement in the load carrying capacity of the new composite pavement was observed. Cores were obtained from the test tracks and tested for shearing strength at the concrete-asphalt interface. The average shearing strength was 1.4 MPa. No cracks related to loading were observed during the loading period.

Despite subjecting the test tracks to relatively small load repetitions, the research project produced valuable information to guide the implementation of the UTW in Florida. Based on the Gainesville research project, it was possible to develop guidelines for design, materials and construction specifications for UTW projects as well as criteria for identifying candidate projects.

THE ELLAVILLE WEIGH STATION

With the success of the research phase, plans were prepared to implement the UTW in projects for rehabilitation of asphalt pavements. Plans were made select a section of roadway that carried significant truck traffic where the existing asphalt pavement had developed chronic and severe rutting problems. The Ellaville Truck Weigh Station provided the ideal site for the application of the UTW.

Located on I-10 in North Florida, the Ellaville Weigh Station includes a 620 m long and 4.8 m wide traffic lane and a 1.2 m shoulder along both sides. The shoulders are widened to 3 m forming a bypass lane around the weighing platform area and the agricultural inspection post. A temporary 7 m wide parking area for trucks is also available east of the weighing platform. The existing pavement was composed of 80 mm to 175 mm asphalt layer on a 275 mm of limerock base, which had been stabilized according to FDOT standards.

More than 1400 trucks travel through the weigh station each day. As they enter the station from I-10, they begin to slow down until coming to a complete stop prior to proceeding on to the weighing platform. After completing the weighing process, the trucks continue in a slow speed toward the agricultural checkpoint. From the checkpoint to the exit the trucks increase gradually reaching a maximum at the merging lane with I-10. This travel pattern had caused a gradual increase in rutting from the entrance reaching maximum levels prior to the weighing platform and then gradually decreasing toward the exit. The rutting ranged from 9 mm to 45 mm. Such severe rutting caused significant roughness of the pavement surface and suggested inadequate load carrying capacity of the asphalt layer. Maintenance effort, by applying thin asphalt overlays, did restore a smooth ride initially. However, rutting resumed within a short time with the daily heavy truck traffic.

It can be argued that a reconstruction with full depth conventional concrete pavement would have been the best solution for the weigh station. However, there are plans to relocate the weigh station further west of its current location to handle the increasing truck traffic on I-10. Thus UTW seemed to be a good interim solution for the chronic rutting problem of the asphalt pavement.

The UTW would restore the ride smoothness and the composite concrete/asphalt layer would enhance the load carrying capacity of the pavement. Also, the high volume of truck traffic would provide an excellent opportunity for an accelerated evaluation of the UTW performance and for determination of its service life. This information would be a valuable input for the life cycle cost analysis of the UTW as a viable option for the rehabilitation of intersections and other medium-volume roads. These and other considerations made the Ellaville Weigh Station an ideal project to implement the first UTW on Florida highways.

Initial Testing and Measurements

A series of tests, condition surveys and measurements were performed at the weigh station to obtain data for the development of design and construction specifications for the UTW. Falling Weight Deflectometer (FWD) tests were performed on the existing asphalt surface to evaluate the condition of the support layers. The average deflection of the asphalt pavement under 550 kPa FWD load was 300 microns. This deflection was 60% lower than the average deflection measured on the asphalt surface at the site of the Gainesville test track (5), and was compatible to deflections obtained from good performing Interstate pavements. It was concluded that the support layers had adequate stiffness to carry the heavy truck traffic and that the support layers did not contribute to the rutting problem of the asphalt layer. No attempt was made to utilize computer models to calculate the modulii of the pavement layers.

Cores were obtained at locations where FWD tests were performed. An inspection of the cores revealed the presence of more than one layer of asphalt. Cracks were noted within the bottom one third of some of the cores. The thickness of each asphalt core was measured. A thickness profile of the existing asphalt layer along the traffic lane and shoulders was established based on thickness measurements from individual cores. The thickness profile showed a variable asphalt thickness ranging from 80 mm to 175 mm in the traffic lane. Samples of asphalt, limerock base, subgrade and embankment, were obtained for laboratory tests. None of the test results indicated any potential problems that would exclude the UTW as an option for pavement rehabilitation.

A condition survey of the asphalt surface revealed significant rutting and moderate cracking. Rut measurements were taken in the general vicinity of the core holes and the FWD test points. The rut depth and asphalt layer thickness data were used to determine the required depth of milling in the asphalt layer. Plate bearing tests were also performed to measure the modulus of the support layers. Results of the plate-bearing test confirmed the conclusions from the FWD tests that the support layers had adequate stiffness.

Design of the UTW

The main objective of the design was to achieve the maximum bond at the interface between the UTW and the asphalt surface. Without an adequate bond, the UTW would fail prematurely.

Based on results from the Gainesville test tracks, it was decided to mill the asphalt surface and use two thicknesses and two joint spacing for the UTW.

Figure 1 shows the design layout of the Ellaville project. The 620 m traffic lane included six sections. Three sections are on the west side of the weighing platform and the remaining three are on the east side. Three main designs were provided in the west section and were repeated in the east section. The three designs included one 80 mm (S1W) and two 100 mm sections (S2W and S3W). The S1W section had a four-panel formation across the lane with 1.2 m joint spacing. The S2W and S3W sections included a three-panel formation with 1.6 m joint spacing, and a four-panel formation of 1.2 m respectively. Section S4E, S5E and S6E mirrored the designs of S3W, S2W and S1W respectively.

The entrance and exit areas had less rutting since the trucks travel at a greater speed. Therefore, S1W and S6E were designed with thinner (80 mm) UTW layers. Close to the weighing platform, the original pavement exhibited maximum rutting caused by the slowing and breaking action of the trucks as they approached the platform. Thus the thickness of the UTW was increased to 100 mm. Three and four panel formations were used, with the four-panel formation being closest to the weighing platform. The thickness for the shoulders, bypass lanes and parking sections was 80 mm. The panel formation in these areas matched that on the traffic lane.

The design also called for milling the entire asphalt surface. The milling was specified to remove the rutting, oily spots from the asphalt surface, and produce a level surface with a rough texture. The maximum depth of the milling was set at 40 mm for the 100 mm UTW sections and 20 mm for the 80 mm sections. The milling depth on the traffic lane was selected to ensure that at least 50 mm layer of asphalt remained after milling. This was based on the thickness profile of the original asphalt layer.

The width of the joint opening was set at 3 mm to 5 mm, requiring only one pass of the joint saw. The depth of the joint groove was specified at one third of the thickness of the UTW section. The transverse joint between the two adjacent sections was designed to be fully cut to completely separate the sections from each other. This design detail was included to avoid the development of cracks, which may have developed as a result of the mismatched alignment of the longitudinal joints and/or layer thickness. These joints were designed to be sealed to prevent water from passing through. Also, a drainage trench was to be constructed beneath the transverse joint along the width of the pavement. The purpose of the transverse drainage was to collect and drive away any water that may penetrate the joint groove. The initial design did not call for the sealing of the other panel joints. However after construction, all joints in the east sections were sealed, using a silicone sealant. The joints in the west sections were left unsealed with the exception of the last 20 m of S3W closest to the weighing platform. The joints in these segments were sealed similar to those in the east sections.

Project Specifications

The UTW project was considered a non-conventional concrete paving operation, requiring special standards and specifications to ensure successful construction. A Technical Special Provisions (TSP) was developed for the project. The TSP included provisions for Fast Track concrete mixtures including fiber in the west sections and plain concrete in the east sections. Other provisions covered construction aspects such as asphalt milling, thorough cleaning of the milled surface, as well as placement, curing and joint sawing of the UTW. The joint sawing requirement called for early saw cutting of joints using green-concrete sawing machines to prevent uncontrolled shrinkage cracks. The TSP also required the use of a slipform paver in the traffic lane. The use of the slipform on the shoulder, bypass and parking areas was optional.

Pavement acceptance was based on three requirements; compressive strength, bond strength and rideability. These requirements were designed to ensure the quality of the end product. The bond strength was evaluated based on the Iowa Shearing Test (6). The specified requirement for the average shearing strength was 1.4 Mpa. The TSP required that 6 cores be obtained from each concrete lot. If the average shearing strength of the 6 cores was less than 1.4 Mpa, the contractor had the option to obtain 3 additional cores. The lowest and the highest test results of the 9 samples would be discarded and an average shearing strength of the remaining 7 samples would be considered for acceptance. A concrete lot constituted 4000 m^2 or one half the paved area in one day. Compressive strength requirements at 24 hours and 28 days were 17.5 MPa and 40 MPa respectively. The 24-hour strength was used as criteria for opening the pavement to traffic. The 28-day test results were used for strength acceptance. The ride acceptance was based on maximum profile index of 110 mm/km as measured by the California Profilograph. The TSP called for penalties if test results failed to meet any of the three acceptance criteria.

Construction

A preconstruction meeting was held between the FDOT and the contractor to discuss, among the issues, the TSP and the design plans for the project. The discussions focused on asphalt milling and surface cleaning to ensure a maximum bond with concrete, the use of a slipform paving machine, rideability requirements, surface finish, texturing, and joint grooving.

The construction started by milling the existing asphalt surface. Table 1 shows the thickness of the asphalt layer within the various sections after milling. In the west sections, the milling machine removed more asphalt than what was specified in the design. In some areas the thickness of the asphalt layer after milling was only 32 mm. At a 20 m segment in S3W just prior to the weighing platform, the depth of the milling left the limerock base exposed. The contractor was notified of the control problem with milling and was asked to remove whatever was left of the asphalt layer in the 20 m segment and place a new layer of asphalt prior to overlay with UTW. He was also asked to adjust the depth of milling, and to strictly adhere to the design plans. The milling resumed on the east sections and was completed in a satisfactory manner. The average thickness of the asphalt layer after milling was 55 mm with many areas having only 30 mm or less asphalt. Areas of very thin asphalt were identified for post construction monitoring under traffic.

A condition survey after milling revealed random and alligator cracking in some areas of the traffic lane and parking. These observations confirmed the results of a visual examination of the core samples of the original asphalt layer. However, since no cracks reflected through the UTW in the Gainesville Tracks (5), it was not anticipated that the cracks would reflect through or affect the performance of the UTW at the weigh station. The milled asphalt surface was broomed and pressure washed to remove debris and dust to ensure a clean and rough surface to achieve a maximum bond between the asphalt and the UTW.

Both fiber and plain concrete were used in the UTW. Table (2) shows the mixture of ingredients at the production stage. The fiber concrete included 1.8 kg/m^3 fibrillated polypropylene (50 mm long) fibers. The concrete mixture used on the west sections contained fibers. Plain concrete was used on the east sections. All ingredients including the fibers were placed and mixed at a concrete batching plant and transported to the project site by truck mixers.

The UTW was placed on the traffic lane using a slipform paver. The asphalt surface was kept in damp condition during the paving operation to prevent a loss of water from the concrete mix, and to possibly improve the UTW/asphalt bond. Paving the entire length of the traffic lane was completed in two days. During the first day, the west sections were paved using fiber concrete. Some difficulties were experienced in controlling the consistency of the fiber concrete mixture. On the second day the east sections were paved with plain concrete. A more desirable consistency was achieved with the plain concrete.

During the paving operation, concrete samples were obtained and tests were performed on the plastic concrete. Table 3 shows the results of slump, air content and unit weight for fiber and plain concrete mixtures. The slipform paver produced best surface finish when the concrete had a slump of 50 to 75 mm. The concrete surface was also finished manually to achieve the smoothest possible surface to meet or exceed specification requirement for ride acceptance.

Surface texturing was followed using a mechanical rake, which produced transverse tines on the pavement surface. After texturing, the pavement surface was covered with a white water-based, curing compound. The TSP also called for wet curing or the Contractor's modified version. The Contractor opted to replace wet curing with a second coating of the curing compound.

The saw cutting of the joints was not completed early enough. Only conventional hard concrete saw machines were available at the project site. This type of machine required that the concrete achieve a fully hardened state. As a result, more than 6 hours passed before the first joint was cut. With the large number of joints in the UTW, and the need to expedite the saw cutting process, every second joint was cut initially using only two saw machines. As a consequence, the joints close to the exit were cut last. The delay in cutting the joints may have caused weakness in the bond or complete debonding in some areas as a result of curling of the concrete layer. Diamond cracking has been observed in the joints close to the exit, which will be discussed in the next section. Green concrete saw machines should have been used, as described in the TSP to cut all the joints while the concrete was in early stages of the hardening process. This would have prevented the concrete curling and debonding of the two surfaces.

On the shoulders, bypass and parking the UTW was placed in fixed forms. A roller vibrating screed was used to compact and finish the surface. This paving operation continued for almost 3 weeks and was not placed with the same quality as on the traffic lane. A similar problem of delayed saw cutting of the joints occurred in these areas. The shoulders/parking areas were placed in relatively warm temperatures. Several uncontrolled cracks on the shoulder and parking panels did develop due to a combination of warm temperatures and a delayed sawing of the joints. This experience demonstrated the importance of using green concrete saw machines to cut joints much earlier than the conventional hard-concrete saw machines. As a general rule, joint sawing should begin as soon as the partially hardened concrete can support the weight of the green saw without causing damage to the concrete surface or raveling of the joint groove.

Results of the Acceptance Tests

The TSP for the project specified three acceptance criteria including Compressive strength, Shearing strength and rideability. During construction, concrete cylinder samples were prepared and tested for Compressive strength at 24 hours and at 3, 7, 28, and 91 days. Flexural and modulus of elasticity samples were also prepared and tested at 28 days for research purposes. Table 4 shows the test results of the hardened concrete. The concrete met the 24-hour and 28-day compressive strength requirements of 17.5 MPa and 40 MPa respectively.

Cores samples were obtained from the fiber concrete and plain concrete sections. By following the TSP requirements, 9 cores were extracted from the west sections and 6 from the east sections. The Iowa-shearing test was performed on the cores. The test results and averages are shown in Table 5. These test results met the *shearing* strength requirement of 1.40 MPa.

As required by the TSP, a rideability acceptance test was performed on the traffic lane. The profile index (PI) on the traffic lane was 45 mm/km, which was less than the required 110 mm/km. The use of slipform paver and added control of the quality of surface finish contributed to excellent ride on the traffic lane surface. In contrast, shoulders, parking, and bypass areas which were paved using a vibratory roller screed. An average PI was 617 mm/km.

Testing and Evaluation of the UTW Project

The weigh station was opened to truck traffic on December 17, 1997. Figure 2 is a view of the Ellaville Weigh Station showing a sample of the heavy truck traffic through the station. On average, more than 1430 trucks pass through the station each day. With such a high traffic volume and truck weights, a close monitoring of the condition of the project began immediately after opening the station to traffic. The Falling Weight Deflectometer tests were performed immediately after construction and after six months from construction.

Falling Weight Deflectometer Tests

The FWD tests were performed on the surface of the UTW at the same test spots as the previous tests on the original and the milled asphalt surfaces. This test pattern provided a direct comparison between the deflections on the three surfaces. Figure 3 shows the range and the average deflection measurements by the seven sensors of the FWD for the three surfaces. Average deflection basins for the three layers is also shown in Figure 3. It is evident that the deflections on the UTW were significantly lower than the deflections on the original and milled asphalt pavements. Table 6 shows the change in deflection in reference to the original asphalt layer as measured by FWD sensor number one. The average deflection was increased by 12 percent after milling the pavement. However, after placement of the UTW, the deflection decreased by 63 percent, suggesting dramatic improvement in the load carrying capacity of the monolithic and composite UTW-asphalt pavement. After six months the FWD tests were repeated on the original test spots. The deflections were still 56 percent lower than what they were on the original asphalt layer. This indicates that the pavement is still acting as a composite of two bonded layers with sufficient support to carrying the high volume and heavy trucks.

Condition Evaluation

As mentioned earlier, late sawing the joints and warm weather contributed to the cracking of several panels in the shoulder and parking area. Panels with hairline cracks were left alone. Those panels that had wide cracks with the potential of spalling were replaced.

The panel replacement operation was simple and relatively quick. A cracked panel was first cut with a saw machine along its four joints to isolate it from the surrounding sound panels. Then the panel was broken up into small pieces with a jackhammer and removed. The condition of the asphalt surface was examined to ensure no damage to the surface. The asphalt surface was cleaned thoroughly, and then resurfaced with fresh concrete. Had the asphalt been severely damaged or completely removed with the broken concrete, then the plan would have been to cut into the base to a depth of at least 25 mm. This would have been followed by thorough compaction of the base and placement of a full depth panel.

Another problem occurred which was not anticipated. Many trucks were coming off and then back on to the shoulder/parking. This caused multiple panels along the embankment to break. Reflector rods were placed along the edges to prevent the trucks from coming off and on to the shoulders. This problem triggered a review of future designs of UTW. For future projects, where there are no curbs and gutters, an asphalt shoulder will be added to the design to provide support to the edge panels of the UTW. In projects where curb and gutter are present, they tend to provide adequate support to the edge panels provided that the surface of the UTW is even with that of the gutter.

Two formal crack surveys were performed just prior to opening the station to traffic on July 23, 1998. Table 7 presents a breakdown of the number of cracked panels according to the UTW section and location (traffic lane, shoulder, bypass or parking). All the cracks which were observed prior to opening to traffic were on the shoulder or parking areas. These were hairline cracks running along the joint groove. Almost all cracks observed on the July 23, 1998 survey were corner cracks. These corner cracks are most likely due to delaminations at the interface between the asphalt surface and the UTW. The most likely cause for of these delaminations is delayed joint sawing, which allowed the concrete layer to curl at a very early age when the bond with the asphalt had not been sufficiently developed. Figure 4 shows diamond shaped cracking formed by corner cracks of four panels. This was a typical cracking pattern on 8 consecutive clusters of panels close to the exit. A review of the record indicated that these panels were the last to be saw cut.

The percentage of cracked panels on the traffic lane is one percent. On the shoulders, bypass and parking the number of cracked panels is 3 percent. This percent of cracking was anticipated, due to some problems during construction. This project is expected to have up to 3% cracked panels per one year. This percent of cracking is still acceptable if one considers 1430 trucks weighing between 10,000Kg and 54,000Kg (with permit) to pass through this station.

A crack survey is performed every 6 months to monitor the pattern and extent of cracking. Also training will be provided to FDOT maintenance personnel on the proper and effective procedure to replace the cracked panels.

Predicted Service Life of UTW

Implementing the UTW on the high volume truck traffic at Ellaville Weigh Station presented an excellent opportunity to predict the useful service life of the UTW on intersections and other medium traffic highways. Based on the information from the weigh station, the average number of trucks passing through per day is 1430. Between December 1997 and July 1998, 272,000 trucks passed through the weigh station. During this period, the total number of (80kN) ESAL was 626,000. Based on current traffic counts of the number of trucks, it is predicted that between August and December, 1998, an additional 215,000 truck will pass through the weigh station.

Based upon these estimates, the total number of ESALs imposed on the UTW during the first year will be 1.12 million.

An additional traffic analysis was performed on a very busy intersection in Tampa. This intersection is located at Kennedy Boulevard and SR 600. The AADT is 37,300 with 5 percent truck traffic. The total number ESALs per year on this intersection are estimated at 230,000. Assuming that this asphalt intersection will be rehabilitated with UTW, it can then be concluded that one year of the service life of the UTW at the weigh station is equivalent to 4.8 years of service life at the intersection.

Since the percent of cracked panels, under 626,000 ESALs (December '97 to July '98), is anticipated to be one percent of the total panels on the traffic lane at the weigh station, one can predict that only 0.4 percent of all UTW panels at the intersection (230,000 ESALs) may crack per one year. The condition of the UTW at the weigh station will be closely monitored to establish, based on actual performance, the most accurate prediction of the service life of UTW projects in general.

CONCLUSIONS

With the success of the UTW research in Gainesville, the FDOT implemented the first UTW project in the rehabilitation of pavements at the Ellaville Truck Weigh-Station on I-10. Based on the observations, tests and evaluations of the Ellaville weigh station project during the period covered in this paper, the following conclusions can be drawn:

1. Design, specification and construction of the UTW should place primary emphasis on achieving maximum bond between the UTW and asphalt surface. Poor bond from delayed sawing of joints or poor surface preparation will ultimately lead to cracking of the UTW panels.
2. Specifying compressive strength, shearing strength at UTW/asphalt interface and rideability as acceptance criteria can produce better performance and ensure smoother ride.
3. Ultra-Thin Whitetopping panels with 1.2 m and 1.6 m joint spacing seem to provide equal performance.
4. The UTW on 32 mm thick asphalt layer did not show any premature cracking. This finding permits wide implementation of UTW in Florida, where the asphalt layers are relatively thin compared to northern states.
5. A preconstruction conference with the contractor is essential to discuss the specific and unique features of the UTW. Since the UTW projects are generally small, it is essential to plan ahead to eliminate any mistakes during construction and ensure maximum success.

6. The effect of fibers on performance of the UTW could not be determined. Sections using plain concrete and those using fiber concrete performed equally well. More truck traffic at the Ellaville project may provide a good opportunity to better assess the impact of the fibers.
7. Sealing the joints did not make any difference in the performance of the UTW sections. Water related distress was not observed on this project despite heavy rains during 1998.
8. A one-year service life of the UTW at the weigh station is equivalent to 4.8 years of service life on a medium traffic intersection. This is based on comparison of the number of ESALs each surface may carry.

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6. Determination of the Shear Strength at the Interface of Bonded Layers of Concrete. AASHTO Designation: TXXX-92 (SHRP #5017), Iowa 406-B (1984).

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Table 1 Layer Thicknesses (mm) for UTW Sections

LAYER	S1W			S2W			S3W		
	Hi	Lo	Avg	Hi	Lo	Avg	Hi	Lo	Avg
Existing Asphalt	115	107	**111**	107	86	**97**	134	86	**110**
Milled Asphalt	90	37	**64**	90	56	**73**	60	32	**46**
UTW	130	86	**108**	110	86	**98**	112	103	**108**

(Layer thicknesses were determined from core measurements.)
Hi = high: Lo = Low: Avg = average

LAYER	S4E			S5E			S6E		
	Hi	Lo	Avg	Hi	Lo	Avg	Hi	Lo	Avg
Existing Asphalt	147	92	**120**	100	N/A	**100**	120	88	**104**
Milled Asphalt	71	55	**63**	75	58	**67**	89		**89**
UTW	129	98	**114**	115	96	**106**	91	83	**87**

Table 2 Concrete Mixture Ingredients

Material Mixture Component	Quantity (kg/m^3) Fiber	 Plain
Type I Portland Cement	362	364
Coarse Aggregate (25 mm size)	984	991
Silica Sand	792	802
Total Water	142	142
Water - Cement Ratio (W/C)	0.39	0.39
Air Entrainment (ml)	73	116
Water Reducer (ml)	1393	1393
Polypropylene Fibers (kg/m^3)	1.8	

(1 lbs/cy = 5.93E-1 ^{3} : 1 in. = 25.4 mm : 1 oz = 29.6

Table 3 Properties of Plastic Concrete

Tests	Fiber	Plain
Slump (mm)	58	52
Entrained-Air (%)	4	4
Unit Weight ((kg/m^3)	2282	2299
Ambient Temp (°C)	22	24
Concrete Temp (°C)	24	25

($T_{°C}$ = ($T_{°F}$ -32)/1.8 : 1 pcf = 16.0 ^{3})

Table 4 Test Results of Hardened Concrete

Property (Mpa)	Age (day) 24-hr	3	7	21	28	91
FIBER CONCRETE						
Compressive (f'c)	19	31	40	-	47	49
Shearing Stress (τ)	-	-	-	-	1.53	-
Flexural (M_R)	-	-	6	-	6	-
Modulus of Elasticity (E)	-	-	-	-	33,50	-
PLAIN CONCRETE						
Compressive (f'c)	31	44	49	56	57	59
Shearing Stress (τ)	-	-	-	-	2.27	-
Flexural (M_R)	-	-	7	-	8	-
Modulus of Elasticity (E)	-	-	-	-	38,00	-

Table 5 Shearing Strength of Hardened Concrete

West Sections (FIBER CONCRETE)

Section	No.	Shear (Mpa)	Remarks
S1W	1	0.99	Debris at interface
	2	1.61	
	3	1.67*	
	4	2.67*	
	5	1.03	Debris at interface
S2W	6	1.41	
S3W	7	0.46	Overmilled/New asphalt layer
	8	1.43	
	9	2.53*	

Note : Highest and Lowest results were excluded and remaining 7 results were averaged. Core dia. = 104 mm

* Additional cores per Technical Special Provisions (TSP)

Average shear strength (Mpa) = 1.52

East Sections (PLAIN CONCRETE)

Section	No.	Shear (Mpa)	Remarks
S4E	1	3.76	
	2	1.89	
S5E	3	1.80	
	4	2.08	Crack in asphalt
S6E	5	1.40	
	6	2.67	

Average shear strength (Mpa) = 2.27

1 in. = 25.4 mm: 1 psi = 6.9 kPa

Table 6 Change in FWD Deflections

Sensor No.	Existing AC	Post Milling	Post UTW Overlay	6 - Month Evaluation
		Average Deflection		
1	307	343 **(12%)**	115 **(-63%)**	134 **(-56%)**

Note:( ) - Percent Change in reference to deflection on Existing AC.

Table 7 Crack Survey Results

Survey Date	**S1W**			**S2W**			**S3W**			
	TLN	SHLD	PRK	TLN	SHLD	PRK	TLN	SHLD	PRK	By-Pass
Dec. 17, 1997										
Jul. 23, 1998	**2**							**8**		**16**

Survey Date	**S4E**			**S5E**			**S6E**			
	TLN	SHLD	PRK	TLN	SHLD	PRK	TLN	SHLD	PRK	
Dec. 17, 1997		**3**	**4**			**5**				
Jul. 23, 1998		**8**	**7**		**8**	**18**	**16**	**5**		
Subtotal	**2**	**11**	**11**	**0**	**8**	**23**	**16**	**13**	**0**	**16**
% Cracked Panel	**0%**	**1%**		**0%**	**1%**		**1%**	**1%**		

% Project Total **TLN**	=	**1%**
SHLD/PRK/By-	=	**3%**

Note: Survey excluded cracked panels at edge of pavement due to encroachment.
TLN = Traffic Lane: SHLD = Shoulders: PRK = Parking
Traffic Lane = 1800 panels: Shoulders/Parking/By-Pass = 2700 panels

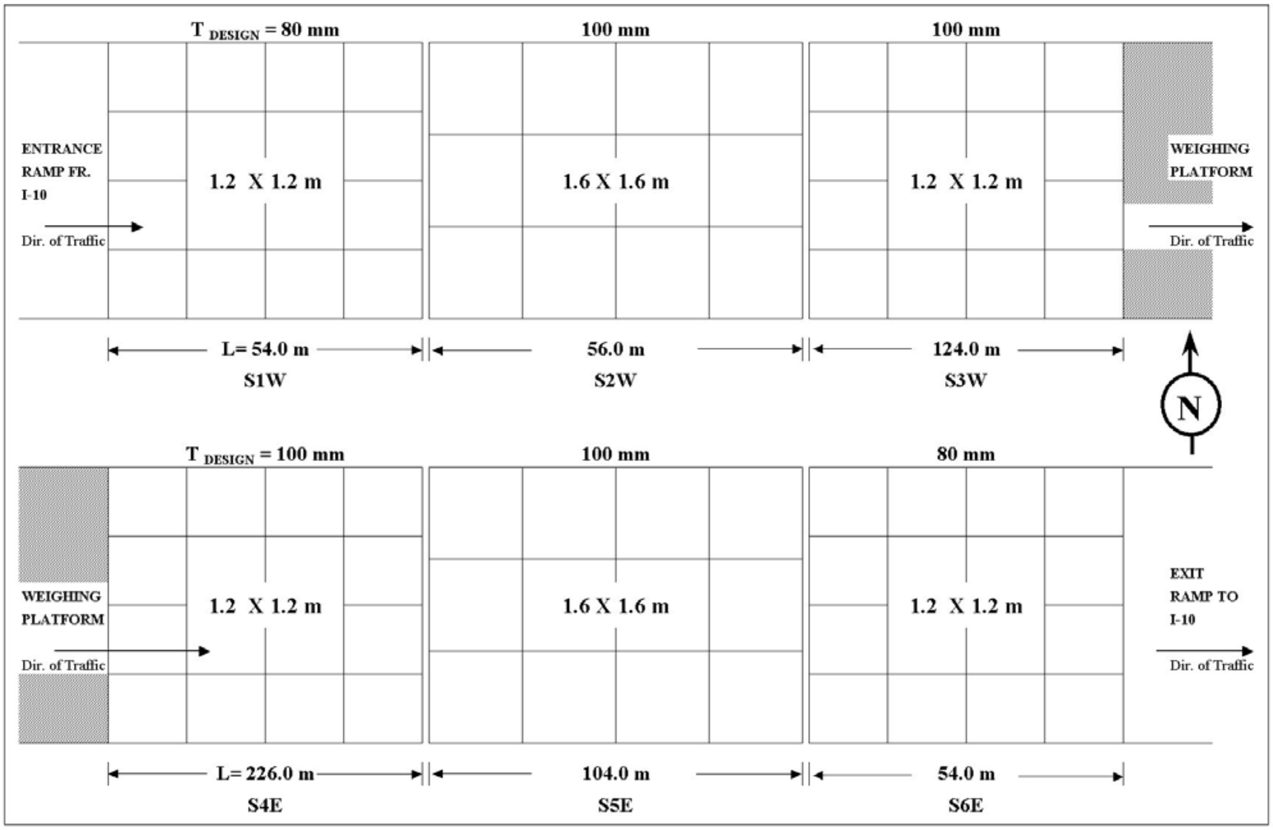

Figure 1 - Design Layout of UTW at Ellaville Weigh Station

Figure 2 - View of the UTW at Ellaville Weigh Station

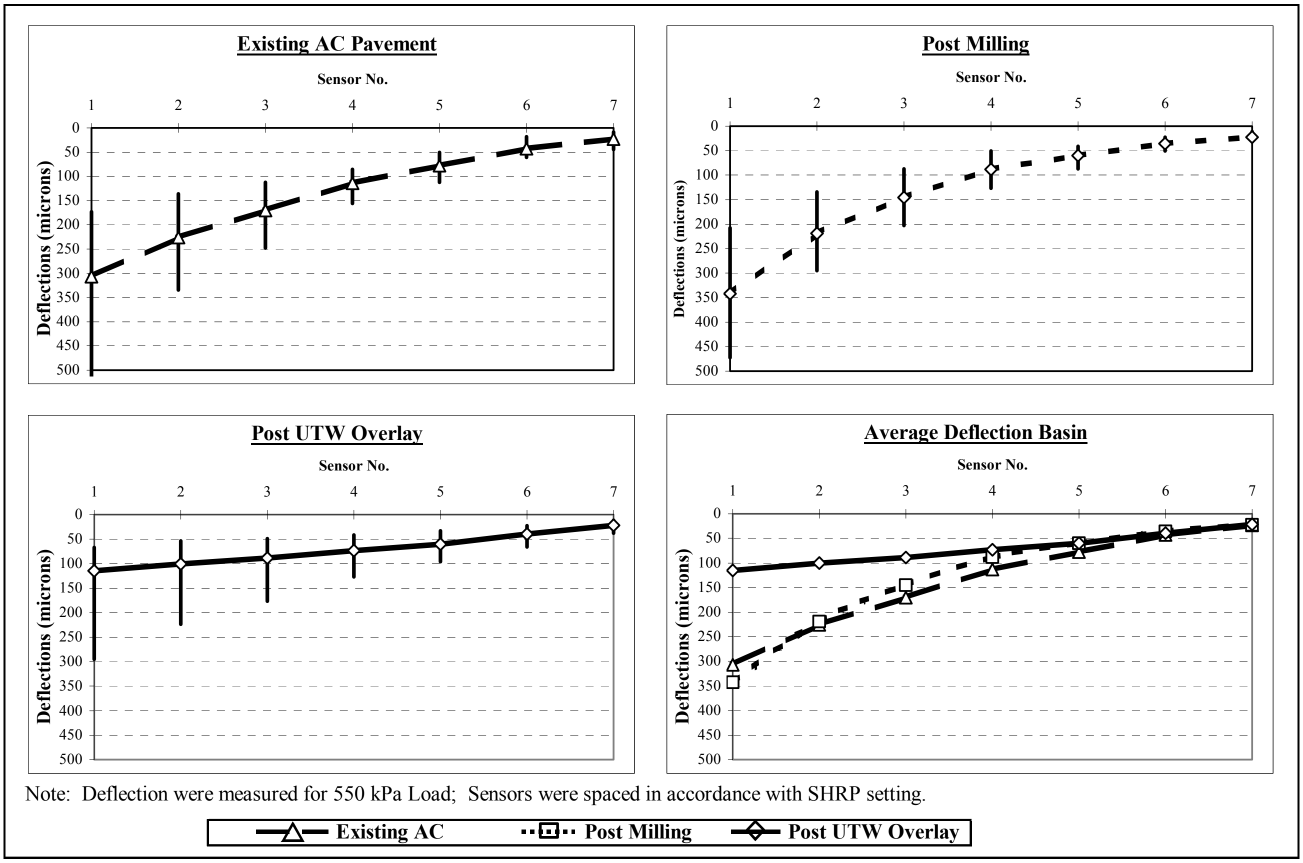

Figure 3 - FWD Deflections

Figure 4 - Diamond Cracking