

PERFORMANCE OF ULTRA-THIN WHITETOPPING IN FLORIDA

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ABSTRACT

Three experimental test tracks were constructed in Gainesville, Florida to evaluate design, construction and performance of the ultra-thin whitetopping (UTW). In Test Track 1, a 100-mm (4-in.) thick UTW was placed on existing pavement composed of 38-mm (1.5-in.) asphalt pavement and 150-mm (6-in.) concrete base. The surface of the asphalt pavement was prepared in three ways: milling, broom cleaning, and applying a thin asphalt crack relief layer (CRL). Test Track 2 consisted of 75-mm (3-in.) and 100-mm (4-in.) thick sections on a 38-mm (1.5-in.) asphalt pavement and a 163-mm (6.5-in.) base layer of sand and Florida limerock material. The surface of the asphalt pavement was prepared in two ways: milling and broom cleaning. In Test Track 3, a 50-mm (2-in.) thick concrete overlay was placed on 38-mm (1.5-in.) milled asphalt pavement and 163-mm (6.5-in.) base layer similar to that in Test Track 2. Joint spacings in Tracks 1 and 2 were 1.22 m X 1.22 m (4 ft X 4 ft) and 1.83 m X 1.83 m (6 ft X 6 ft). Test Track 3 had 0.92 m X 0.92 m (3 ft X 3 ft) and 1.22 m X 1.22 m (4 ft X 4 ft) joint spacings. Polypropylene fibers were used in Tracks 1 and 2. Polyolefin fibers were used in Track 3. High early strength concrete was designed for the UTW. Bond strength at the concrete-asphalt interface was generally above 1.4 MPa (200 psi). The test tracks were subjected to approximately 60,000 (18-kip) ESALs using a truck loaded with concrete blocks. Falling Weight Deflectometer (FWD) test results showed significant improvement in the structural capacity of the pavement after placement of the UTW. Frequent condition surveys indicated excellent performance of the three test tracks. The UTW is a viable option for rehabilitation and restoration of asphalt pavements.

Keywords: concrete, ultra-thin whitetopping; design; rehabilitation; fibers; high early strength, Falling Weight Deflectometer.

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 mm¹ (5 in.). In Florida, a whitetopping project was constructed on US 1 in 1987. Thicknesses for the concrete overlay included 150 mm (6 in.), 175 mm (7 in.) and 200 mm (8 in.). The project has shown good performance and has developed very low faulting and cracking.

During the past seven years, many state highway agencies, counties and cities have placed much thinner concrete overlays than the conventional whitetoppings (1). These concrete overlays have been commonly referred to as ultra-thin whitetopping (UTW). They have ranged in thickness from 50 to 100 mm (2 to 4 in.). Following the encouraging results from the first experimental project in Louisville, Kentucky, over 70 UTW projects have been built 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 constructions have been developed based on experience from individual projects (3). The rutted asphalt pavements in urban areas are likely to be milled and then overlaid with concrete. Thicknesses

¹ All metric units in the report are converted from English units by using hard conversion.

for the UTW range from 50 to 100 mm (2 to 4 in.) and pavement joint spacings range from 0.61 to 1.83 m (2 to 6 ft). The recommended minimum thickness of the asphalt layer has been 50 to 75 mm (2 to 3 in.) (3). Leaving 50 to 75 mm (2 to 3 in.) of the asphalt in-place, after milling, prevents deterioration and structural damage of the base layer, and insures good bond between the asphalt and the UTW.

Early indications from pavement condition surveys suggest good performance for most UTW projects. (Correspondence with L. Cole dated September 27, 1997.) Design parameters such as overlay thickness, joint spacing, asphalt thickness, base stiffness and bonding between the UTW and asphalt base have significant impact on the success or failure of the UTW. However, there is no analytical procedure to determine critical stresses for individual or a combination of parameters (4). As a result, no formal design procedure is available to account for all the possible conditions which may exist in a potential UTW project. This leaves many unanswered questions about the proper selection of design parameters such as thickness of the asphalt, base stiffness and joint spacing. For example, milling a relatively thin asphalt pavement to remove excessive rutting may leave less than the recommended 50 to 75 mm (2 to 3 in.) asphalt layer beneath the UTW.

The impact of a thinner base is not known. While it has been suggested that shorter joint spacing such as 0.61, 0.92, and 1.22 m (2, 3 and 4 ft) may produce better results, the issues of cost and the introduction of multiple sharp corners in the concrete overlay become important considerations. The base stiffness plays an important role in the decision to select the UTW as the rehabilitation option. Weak base layers which contributed to the original distress may also cause failure of the UTW. Criteria has to be developed for suitable bases for UTW. This requires determination of base stiffness from nondestructive test methods such as the FWD and back calculation computer models.

Other factors which require careful consideration are materials, concrete mixtures and construction techniques. Specific issues facing designers and construction engineers 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.

Realizing the benefits of UTW as a viable rehabilitation/restoration technique and facing many unanswered design and construction questions, the Florida Department of Transportation (FDOT) embarked on a research project to evaluate design, materials, construction and performance of the UTW.

Three UTW test tracks were constructed on a vacated maintenance yard located behind the FDOT State Materials Office in Gainesville. These tracks were loaded for more than eighteen months using trucks carrying concrete blocks. The test tracks were subjected to over 60,000 (18-kip) ESALs. The conditions of the tracks were closely monitored during construction and subsequent loading. Tests using the FWD were performed on the original asphalt pavement and on surface of the UTW. Cores were extracted from the test tracks and tested for bond strength at the concrete-asphalt interface. This paper describes the design, construction, and load-testing of the test tracks. Findings from the FWD tests and condition monitoring of the tracks are also discussed.

TEST TRACKS

Design Concept

Two factors which have significant effect on the long term performance of an UTW are bonding at the concrete-asphalt interface and condition of the structural support of the subbase layer beneath the asphalt. For the purpose of consistency, the asphalt layer beneath the UTW will be referred to as the base and the original base beneath the asphalt layer will be referred to as the subbase. The UTW can be successful only if the subbase is not the source of the distress in the original asphalt pavement. Thicker concrete overlays should be considered if the subbase has exhibited poor structural support by contributing to the deformation and/or cracking of the original asphalt pavement.

Bonding between the concrete overlay and the asphalt base is essential for long term performance of the UTW. Without adequate bonding, the UTW layer will respond independently to the applied loads and will likely develop excessive stresses to the point of failure. Also, temperature curling will tend to lift the corners of the concrete panels (small slabs), giving rise to tensile stresses and expediting the cracking of the UTW. Proper bonding of concrete-asphalt layers can resist curling forces responsible for lifting of panel edges and corners.

The key to a successful design is to ensure bonding by proper surface treatment of the asphalt base. Effective surface preparation for asphalt includes milling and/or thorough cleaning. Milling will remove ruts, oil residue and other surface

distortions, and will provide a rough surface texture which ensures adequate bonding. Thorough cleaning will be effective in cases of pavements that already have rough surface textures (e.g. friction courses) and have not developed ruts deeper than 3 mm (1/8 in.). Another surface treatment includes applying a crack relief layer (CRL) which consists of a thin coat of asphalt and aggregates to prevent reflective cracking. This is a common practice in Florida when resurfacing asphalt pavement. The CRL will also a level surface with a rough texture to allow a good bond with the concrete overlay.

Joint spacing plays an important role in the performance of the UTW. Design of short joint spacing will minimize curling and significantly reduce curling stresses. A combination of short joint spacing and adequate bond will create a composite-monolithic layer of concrete and asphalt which will possess greater load carrying capacity. It has been suggested that under such conditions the concrete layer will likely be permanently in compression (5 and 6). This hypothesis still needs to be confirmed by modeling and field verification of performance of existing UTW projects.

Design Details

Plan and cross-section views of Test Tracks 1, 2 and 3 are shown in Figures 1, 2 and 3, respectively. Test Track 1 is 18.29-m (60-ft) long, including two 1.83-m (6-ft) ramps and consists of 100-mm (4-in.) thick concrete overlay. It includes a combination of 1.22 m X 1.22 m (4 ft X 4 ft) and 1.83 m X 1.83 m (6 ft X 6 ft) joint spacings (panels). Track 2 is 43.89-m (144-ft) long, including two 1.83-m (6-ft) ramps. This track consists of two sections with overlay thicknesses of 75 and 100 mm (3 and 4 in.). The joint spacing patterns in each subsection include 1.22 m X 1.22 m (4 ft X 4 ft) and 1.83 m X 1.83 m (6 ft X 6 ft). A 3.66 m X 3.66 m (12 ft X 12 ft) transition slab was constructed between the 75 and 100 mm (3 and 4 in.) sections. Test Track 3 is 14.63-m (48-ft) long, including two 1.83 m (6-ft) ramps. The concrete overlay is 50-mm (2-in.) thick and includes 0.92 m X 0.92 m, 1.22 m X 1.22 m and 3.66 m X 3.66 m (3 ft X 3 ft, 4 ft X 4 ft and 12 ft X 12 ft) panels.

The test tracks were constructed in a maintenance yard located behind the FDOT State Materials Office. In the area where Track 1 was constructed, the asphalt covers an old 150-mm (6-in.) concrete pavement which used to be the floor for aggregate bins. The UTW in Track 1 is supported by a nominal 25-mm (1-in.) asphalt base and a 150-mm (6-in.) concrete subbase. In the area of Tracks 2 and 3 the pavement has a nominal 38-mm (1.5-in.) asphalt thickness (the thickness may actually vary from 25 to 50 mm). The subbase beneath the asphalt pavement in areas where Tracks 2 and 3 were constructed consists of nominal 163-mm (6.5-in.) stabilized layer of Florida limerock and sand. The limerock is soft limestone commonly used as base material beneath asphalt pavement. Tracks 2 and 3 are also supported by 38 mm (1.5-in.) asphalt base but with a stabilized subbase of sand and limerock. The stabilized subbase in Tracks 2 and 3 is substantially weaker than the concrete subbase in Track 1. It should also be noted that this subbase is not representative of typical and much stronger standard bases used in Florida roadways.

Prior to overlay, the surface of the asphalt pavement was prepared using several treatments. In Track 1, three treatments were implemented as shown in Figure 1. The surface treatments included a 10.97-m (36-ft) long section of asphalt CRL, a 3.66-m (12-ft) section of broom-cleaned surface without further treatment, and a 3.66-m (12-ft) section of milled and broom-cleaned surface. The asphalt surface beneath Track 2 was divided into three sections. Two 10.97-m (36-ft) sections at both ends of the track were milled, and the remaining 18.29-m (60-ft) section in the middle was broom-cleaned and left without further treatment. The entire asphalt surface in Track 3 was milled and broom-cleaned to ensure adequate bond, considering the extremely thin 50-mm (2-in.) concrete overlay as shown in Figure 3.

SITE INVESTIGATION AND PREPARATION

Preconstruction Tests

The sites for the three tracks were chosen in three separate locations within the maintenance yard to ensure placement of one of the tracks on concrete (strong) subbase, and the remaining two tracks on stabilized (weaker) subbases. The layout of the tracks was also designed to facilitate movement for loading trucks within the confined area of the maintenance yard. Random cracks existed in the areas where the three tracks were to be constructed. Wide reflective cracks in the base were present in the original pavement surface in Track 1. Cores obtained from the site of Track 1 suggested that these cracks originally developed in the concrete base and then reflected through the thin 38-mm (1.5-in.) asphalt pavement. The reflective cracks were random suggesting no joints in the concrete subbase. The cracks were carefully mapped to determine their effect (if any) on the condition of the UTW after the construction and loading phases.

After selecting sites for the three tracks, the structural support of the existing pavement at the sites of the three tracks was evaluated by performing FWD tests. The three sites were also cored to verify layer thicknesses and to determine material properties of the asphalt, and the stabilized and concrete subbases. Thickness of the asphalt pavement ranged from 38 to 50 mm (1.5 to 2 in.). The thickness of the concrete subbase was approximately 150 mm (6 in.). No distinct boundary was detected for the stabilized subbase layer. However, from best estimates, the subbase thickness is assumed to be 163 mm (6.5 in.). The subgrade is composed of well compacted natural materials.

The asphalt and concrete layers beneath Track 1 represent a strong support system comparable, if not exceeding, the stiffness of most support systems beneath pavements in south Florida. Test Tracks 2 and 3 are being supported by a weaker subbase. This subbase is not considered standard for Florida roadways. Such diverse base/subbase conditions presented an excellent opportunity to closely evaluate UTW design on different support systems. Success of the UTW under the two diverse base/subbase conditions makes it a viable option in the rehabilitation of in-service pavements.

Surface Preparation

Establishing a good bond between the UTW and the asphalt base was the primary objective in the surface preparation phase. Three types of surface preparations were designed as shown in Figures 1, 2 and 3. At sections where the existing asphalt surface was not modified, the surface was thoroughly swept with a street sweeper to remove dust and other debris.

The 10.97-m (36-ft) section of CRL in Track 1 was constructed by first applying a 1.26 L (1/3 gallon) of AC-5 asphalt per square yard of existing surface. This was followed by spreading a layer of graded aggregates 19 mm (3/4 in.) nominal size on the asphalt. Pneumatic rollers were used to firmly embed the aggregates in asphalt and establish a rough surface texture to bond to the concrete overlay. Milled sections of Tracks 2 and 3 were prepared using a 300-mm (12-in.) milling machine. Approximately 6 mm (1/4 in.) of the existing asphalt pavement was milled. All surfaces were swept prior to placement of the UTW. Shear test to evaluate bond strength between different asphalt surfaces and the UTW will be discussed in the following section.

CONSTRUCTION AND MATERIAL TESTING

Mixture Design

During the planning phase, several trial mixtures were batched including plain concrete and concrete with fiber. It was decided to design high early strength (HES) concrete mixtures to allow Fast Track paving. Criteria selected for the concrete mixture included the use of fibers, low water-cement ratio (W/C) and good workability for fiber concrete by using the High Range Water Reducers (HRWR), if necessary. The HES concrete allows early opening of pavements to traffic. Maintaining a workable mixture assures successful placement and finishing of the UTW. The mixture designs for the three test tracks are shown in Table 1.

Cement type I and II were used with and without fly ash. Three aggregate sizes were included. Track 1 used 25 mm (1 in.) nominal size aggregate. Track 2 used 19 mm (3/4 in.) nominal size aggregate. For Track 3, a 9 mm (3/8 in.) size aggregate was used because of 50-mm (2-in.) thick overlay. The W/C for Tracks 1, 2, and 3 were 0.38, 0.38 and 0.39, respectively. High dosage rate of HRWR was used to overcome the loss of workability from the addition of fibers. Fibrillated polypropylene fibers (50 mm long) were used in the mixtures for Tracks 1 and 2 at a rate of 1.8 kg/m³ (3 lb/yd³) of concrete. Polyolefin fibers were used at a dosage rate of 11.86 kg/m³ (20 lb/yd³) for Track 3. The plastic properties of three concrete mixtures were tested at the construction site and are shown in Table 2. The ambient temperatures were similar for the three tracks. Slump values represent the concrete prior to the addition of the HRWR.

Construction

The concrete was delivered in truck mixers. For Tracks 1 and 2, plain concrete mixture was batched at the concrete plant and then transported to the site by truck mixers. Upon arrival at the site of Tracks 1 and 2, the slump was checked and workability increased by addition of HRWR. The fibers were then added to the truck mixer at a dosage rate of 1.8 kg/m³ (3 lb/yd³). The concrete was re-mixed thoroughly to disperse the fibers. For Track 3, polyolefin fibers were added to the concrete at the plant, using a dosage rate of 11.86 kg/m³ (20 lb/yd³). All adjustments to the slump, prior to and after the addition of fibers, were performed at the concrete plant. The concrete was brought to the site of Track 3 ready to be discharged.

Slump tests were performed prior to and after the addition of HRWR and fibers. It should be noted that the slump values presented in Table 2 represent the concrete workability prior to the addition of the HRWR. It was necessary to add HRWR to produce a high initial slump of 125 to 175 mm (5 to 7 in.) since the fibers tend to reduce the slump by 38 to 50 mm (1.5 to 2 in.). A low initial slump would have resulted in a harsh non workable concrete.

Prior to concrete placement, all asphalt surfaces were sprayed with water to keep the surface in a damp condition during concrete placement. In each track, a control section was placed using plain concrete without the addition of fibers. The remaining section was placed using fiber concrete as shown in Figures 1, 2, and 3. The concrete was distributed and compacted in the form using immersion vibrators and vibratory screed.

After placement, the concrete surface was broom-finished in Track 1 and turf-finished in Tracks 2 and 3. White water pigment compound was used for initial curing. Two to three hours after concrete placement, the concrete was sawed to form the joint grooves. Early saw cutting machine was used. This machine uses a special saw blade which cuts through relatively fresh concrete without the need for water. The depth of the joint grooves was 1/3 of the overlay thickness. The width of the saw cut was 3 mm (1/8 in.). The joints could have been cut sooner; however, there was a delay in the setting time of the concrete as a result of the use of large dosages of HRWR to achieve proper workability for construction. The compressive strength was tested at the time of saw cutting of the joint grooves. The strength of concrete was 5.5 MPa (800 psi). At this strength level, the concrete can carry the load of the saw machine and will allow cutting the joint grooves without raveling or distortion of the concrete surface. The joint pattern for the three tracks are shown in Figures 1, 2 and 3. After all the joints were cut, the pavement surface for each track was covered with plastic sheets, kept wet using soaker hoses and allowed to cure for three days.

Testing Hardened Concrete

Concrete samples obtained during placement of the three tracks were tested for compressive, tensile and flexural strengths. The modulus of elasticity (E) for each concrete was also measured. Table 3 shows average values of at least three test results obtained at different ages. It should be noted that all test samples were fabricated from concrete containing polypropylene or polyolefin fibers.

The compressive strength was tested eight hours after placement. This was to evaluate strength development of the HES concrete. The compressive strength was also tested at 24 hours and 3, 7, 28 and 91 days. It should be noted that samples were tested at time of saw cutting the concrete to determine strength of concrete at this event.

The 24-hr strength for Tracks 1 and 2 exceeded 21 MPa (3000 psi). For Track 3, the 24-hr strength reached only 14 MPa (2000 psi). This can be attributed to relatively large dosage of the HRWR which delayed concrete setting and strength development. However, at 28 days the strength of concrete in Tracks 1 and 3 were very close, as shown in Table 3.

The split tensile and flexural strengths, and modulus of elasticity samples were tested at 28 days. The flexural strength at 28 days is very high. The fibers may have contributed to the high flexural strength, and may also have contributed to the variability of these results as shown in Table 3.

Shear tests were performed according to the Iowa shear test method (7). During the 18-month testing period, more than 30 core samples were obtained from different sections, thicknesses and surface treatments. Table 4 shows average shear strength representing the three asphalt surface treatments. There were failures (layer separation) in some samples during the coring operation. However 25 samples were extracted intact and were subsequently tested. The test results indicate that the milled and broom-cleaned surfaces develop a strong bond with the UTW. Based on these test results and excellent performance after 60,000 (18-kip) ESALs, a shearing strength of 1.4 MPa (200 psi) is recommended for quality assurance purposes on Florida's UTW construction projects. Also, it would not be recommended to use the CRL as surface treatment, due to poor shearing strength results.

PAVEMENT TESTING

The test tracks were subjected to truck loading over a period of 18 months. Loading of the test tracks began seven days after construction. Several trucks were used with different sizes and wheel configurations. The most frequently used loading truck weighed 55,700 pounds. Traffic counters were placed at the end of each track for an accurate count. Over the 18-month loading period, the test tracks were subjected to more than 60,000 (18-kip) ESALs. The number of applied 18-

the 18-month loading period, the test tracks were subjected to more than 60,000 (18-kip) ESALs. The number of applied 18-kip ESALs is relatively low compared to in-service roadways. It should be noted that volunteers from FDOT were used to drive the loading trucks. Based on this fact, subjecting the test tracks to 60,000 (18-kip) ESALs in the confined area of the experimental project is considered a major achievement. Truck loading simulated loading conditions on actual highways. This allowed true evaluation of the structural capacity of the test tracks, and provided the opportunity to observe any changes in the condition of the pavement throughout the loading period.

Falling Weight Deflectometer Tests

The existing asphalt pavement was tested using the FWD and Dynaflect prior to placement of the UTW. Immediately after construction and prior to loading, FWD tests were performed on the surface of the UTW at approximately the same positions as those tested on the existing asphalt surface. This was to determine level of improvement in the load carrying capacity of the pavement after placement of the UTW.

Figure 4 shows the FWD deflection basins for the three test tracks prior to and after the placement of the UTW. The deflections in Figure 4 correspond to FWD dynamic load of 550 kPa (80 psi). Significant improvement can be observed in the structural capacity of the pavement as a result of the concrete overlay. The values of the deflections at the six FWD sensors are also presented in Table 5. The table also shows the percent reduction in deflection after placement of the UTW. It can be observed that the 100-mm (4-in.) sections in Tracks 1 and 2 produced the greatest reduction in the maximum deflection (78 percent), indicating significant improvement in the structural capacity of the pavement system. The 50-mm (2-in.) section in Track 3 produced the least percent reduction (44 percent) in the maximum deflection. However, when the maximum deflection and the deflection basin were compared with deflections on conventional 225-mm (9-in.) slab 3.66 m X 6.10 m (12 ft X 20 ft) the structural capacity of the two pavement systems is very comparable.

During the loading period, the three tracks were tested four times using the FWD to evaluate their structural capacity. Deflections were measured at three FWD load levels: 550 kPa, 750 kPa and 950 kPa. Only test results for 550 kPa are presented in this paper. The tests were performed during the early morning hours (AM) and in the mid-afternoon (PM) to determine the extent of curling and evaluate the condition of the bonding between the UTW and the asphalt base. Significant change in deflections between early morning and afternoon would suggest loss of bond which would allow large curling in the concrete panels. Also, significant increase in deflections from one FWD test date to another would indicate degradation in the structural capacity of the pavement system as a result of increased loading cycles. Table 6 and 7 show deflection data from the FWD testing performed in April and August of 1996, respectively. The deflection values in Tables 6 and 7 correspond to 550 kPa load. The deflections in each FWD test cycle were generally low. The change in deflections from one test series to another was not significant enough to suggest deterioration in the structural capacity despite the continuation of the truck loading. With few exceptions, deflections obtained in the AM and PM were only slightly different, indicating good bond and minimum, if any, curling of the concrete panels. A mechanistic model may provide a better tool for analysis of stresses induced by truck and FWD loading.

Condition Monitoring

Visual inspection of the test tracks was performed on a weekly basis. The inspection included monitoring the progress of cracking at the joint grooves after construction and recording any panel cracking or joint spalling. The first joint groove that cracked through was at mid length. Then the joints at quarter lengths developed cracks. During the loading process, 80% of all control joints did crack. However, many of these joints formed only hairline cracks and did not open further during the loading process.

After 60,000 (18-kip) ESALs, only one visible crack was detected in a 1.22 m X 1.22 m (4 ft X 4 ft) panel in Track 1. This panel was subsequently replaced. In Track 3, the 3.66 m X 3.66 m (12 ft X 12 ft) slab cracked into four 1.83 m X 1.83 m (6 ft X 6 ft) panels. This was expected for a 50-mm (2-in.) thick overlay. This cracking pattern may also suggest that 1.83-m (6 ft) joint spacing should be used as the maximum joint spacing for the UTW. Few corners of the 1.22 m X 1.22 m (4 ft X 4 ft) panels were slightly chipped. The corner chipping may have been caused by dislodging of an aggregate particle during early saw cutting of the joint grooves. Also, these corners happened to be in the wheelpath of the loading trucks. A possible solution is to increase the joint spacing to 1.52 m X 1.52 m (5 ft X 5 ft) or 1.83 m X 1.83 m (6 ft X 6 ft) to move the corners away from the wheelpath.

Sections using fiber concrete performed equally well as those using plain concrete. It was not possible to establish the impact of fibers in concrete on the performance of the UTW. More load repetitions may be needed to determine the effect, if any, of fibers on performance.

IMPLEMENTATION OF THE UTW

The UTW experiment is considered a success. This fact is based on the FWD test results and is confirmed by the excellent condition of the three test tracks, despite heavy loading. The very good performance of the UTW in Florida complements similar results in other states. The FDOT is currently developing guidelines and specifications to be used in the implementation of the UTW technique in the rehabilitation of asphalt pavement. However, there are still issues that require further study. These issues include whether or not to seal the joints, role of fibers in the performance of the UTW, and how to assure adequate and permanent bond. The loading of the tracks may continue to obtain as much data as possible.

CONCLUSIONS

After application of 60,000 (18-kip) ESALs, Florida's UTW test tracks are in very good condition. Based on results of this experimental project the following conclusions can be drawn:

1. The UTW significantly improves the load carrying capacity of asphalt pavements. It can be considered as viable option for rehabilitation and restoration of asphalt pavements.
2. Adequate bond between the UTW and asphalt base is essential for the long term performance of the UTW. The shearing strength at the UTW-asphalt interface should be at least 1.4 MPa (200 psi), when the Iowa shearing test is performed. This is based on 25 test results and excellent performance of the test tracks.
3. The UTW should be designed using short joint spacings. Based on results of this study, the preferred joint spacing for 75 and 100-mm (3 and 4-in.) thick UTW should be 1.22 to 1.83 m (4 to 6 ft). For a 50-mm (2-in.) thick UTW, the joint spacing should not be greater than 1.22 m (4 ft).
4. The UTW performed well on 38-mm (1.5-in.) asphalt base. This finding allows the use of the UTW in Florida where the asphalt layers are relatively thin compared to northern states.
5. 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 load repetitions may be needed to establish the effect, if any, of the fibers on the performance.

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Table 1 Concrete Mixture Proportions

Material (lbs/cy)	Track 1	Track 2	Track 3
Type II Portland Cement		750	
Type I Portland Cement	590		760
Type F Fly Ash	150		
Coarse Aggregate (Nominal Size)			
#57 Crushed Limestone (1 in.)		1800	
#67 Crushed Limestone (3/4 in.)	1750		
#89 Crushed Limestone (3/8 in.)			1540
Silica Sand	1010	1070	1400
Total Water	280	285	295
Water - Cement Ratio (W/C)	0.38	0.38	0.39
Ordinary Air Entrainment (oz.)	21	4	0.5
Ordinary Water Reducer (oz.)	23	5	15
High Range Water Reducer (HRWR) (oz.)	120	120	137
Polypropylene Fibers (lbs/cy)	3	3	
Polyolefin Fibers (lbs/cy)			20

(1 lbs/cy = 5.93E-1 kg/m³ : 1 in. = 25.4 mm : 1 oz = 29.6 ml)

Table 2 Plastic Properties

Tests	Track 1	Track 2	Track 3
Slump (Prior to HRWR)* (in.)	2.25	3.75	3.5
Entrained-Air (%)	2.5	N/A	1.6
Unit Weight (pcf)	N/A	144.3	141.6
Ambient Temp (°F)	82	72	83
Concrete Temp (°F)	83	78	80

(Note: Addition of fibers reduced slump by 1.5 - 2.0 inches.)

($T_c = (T_f - 32)/1.8$: 1 pcf = 16.0 kg/m³)

* HRWR was added to increase the slump prior to addition of fibers.

Table 3 Average Test Results of Hardened Concrete

Property (psi)	Age (day)					
	8-hr	24-hr	3	7	28	91
Track 1						
Compressive (f_c)	1530	3830	5320	5900	7430	N/A
Tensile (f_T)	-	-	-	-	620	-
Flexural (M_R)	-	-	-	-	1000	-
Modulus of Elasticity (E)	-	-	-	-	4.9 E+6	-
Track 2						
Compressive (f_c)	2400	3510	5950	7070	8140	8580
Tensile (f_T)	-	-	-	-	660	-
Flexural (M_R)	-	-	-	-	800	-
Modulus of Elasticity (E)	-	-	-	-	4.7 E+6	-
Track 3						
Compressive (f_c)	470	2000	4590	6370	7350	7960
Tensile (f_T)	-	-	-	-	620	-
Flexural (M_R)	-	-	-	-	900	-
Modulus of Elasticity (E)	-	-	-	-	4.2 E+6	-

(Note: All results were obtained from fiber concrete specimens. Avg. of three (3) samples)
(1 psi = 6.9 kPa)

Table 4 Average of Shear Test Results (psi)

Track	UTW Thickness (in.)	Asphalt Surface Treatment		
		Cleaned	Milled	Crack Relief Layer (CRL)
1	4	150	350	110
2	3	360	380	-
2	4	240	430	-
3	2	-	340	-

Note: Average Results were from atleast three (3) samples.
Tested in accordance with IOWA SHEAR TEST Procedure - IOWA 406-B (1984)

Table 5 Comparison of FWD Deflections on Asphalt and UTW Surfaces.

TRACK 1 4 in. Section			
Sensor No.	Deflection (microns)		Percent Reduction in Deflection
	Existing AC Surface	UTW Surface	
1	615	131	78%
2	464	109	76%
3	345	89	74%
4	236	69	71%
5	175	51	71%
6	98	38	61%

TRACK 2 3 in. Section			
Sensor No.	Deflection (microns)		Percent Reduction in Deflection
	Existing AC Surface	UTW Surface	
1	909	457	50%
2	536	277	48%
3	351	222	37%
4	219	137	37%
5	154	70	55%
6	93	51	45%

TRACK 2 4 in. Section			
Sensor No.	Deflection (microns)		Percent Reduction in Deflection
	Existing AC Surface	UTW Surface	
1	936	202	78%
2	601	238	60%
3	386	204	47%
4	220	150	32%
5	136	100	26%
6	87	71	20%

TRACK 3 2 in. Section			
Sensor No.	Deflection (microns)		Percent Reduction in Deflection
	Existing AC Surface	UTW Surface	
1	749	420	44%
2	487	297	39%
3	331	167	50%
4	212	100	53%
5	153	60	61%
6	94	42	55%

(1 micron = 0.001 mm)

Table 6 Falling Weight Deflectometer Test Results on April 18, 96 - AM vs. PM (microns)

Test	Track 1			Track 2			Track 3		
	AM	PM	% Change	AM	PM	% Change	AM	PM	% Change
Positions	Entire Track								
Center Panels	120	133	11	246	261	6	415	399	-4
Edge Panels	163	166	2	439	322	-27	579	501	-13
1.22 m X 1.22 m (4 ft X 4 ft) Panels									
Center Panels	119	123	4	263	267	1	385	395	3
Edge Panels	163	166	2	330	289	-12	429	433	1
1.83 m X 1.83 m (6 ft X 6 ft) Panels									
Center Panels	135		19	226	255	13			
Edge Panels	219	212	-3	583	386	-34			
0.91 m X 0.91 m (3 ft X 3 ft) Panels									
Center Panels							405	416	3
Edge Panels							570	541	-5

Table 7 Falling Weight Deflectometer Test Results on August 1, 96 - AM vs. PM (microns)

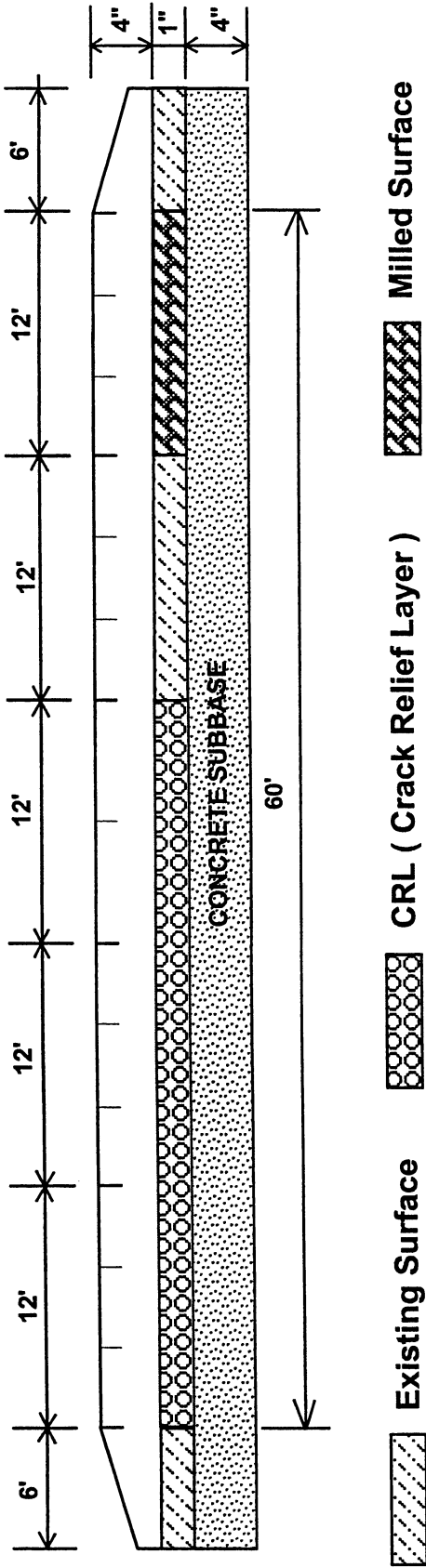
Test	Track 1			Track 2			Track 3		
	AM	PM	% Change	AM	PM	% Change	AM	PM	% Change
Positions	Entire Track								
Center Panels	151	185	22	226	290	28	405	366	-10
Edge Panels	190	192	1	325	265	-19	413	401	-3
1.22 m X 1.22 m (4 ft X 4 ft) Panels									
Center Panels	150	178	18	232	270	16	452	392	-13
Edge Panels	190	192	1	309	260	-16	373	333	-11
1.83 m X 1.83 m (6 ft X 6 ft) Panels									
Center Panels	155	214	38	214	335	57			
Edge Panels	215	235	9	358	275	-23			
0.91 m X 0.91 m (3 ft X 3 ft) Panels									
Center Panels							389	334	-14
Edge Panels							407	426	5

- Notes: 1) FWD Deflections represent average values for all test positions.
 2) Test positions on panels along the center and edge of the tracks.
 3) FWD Load = 550 kPa

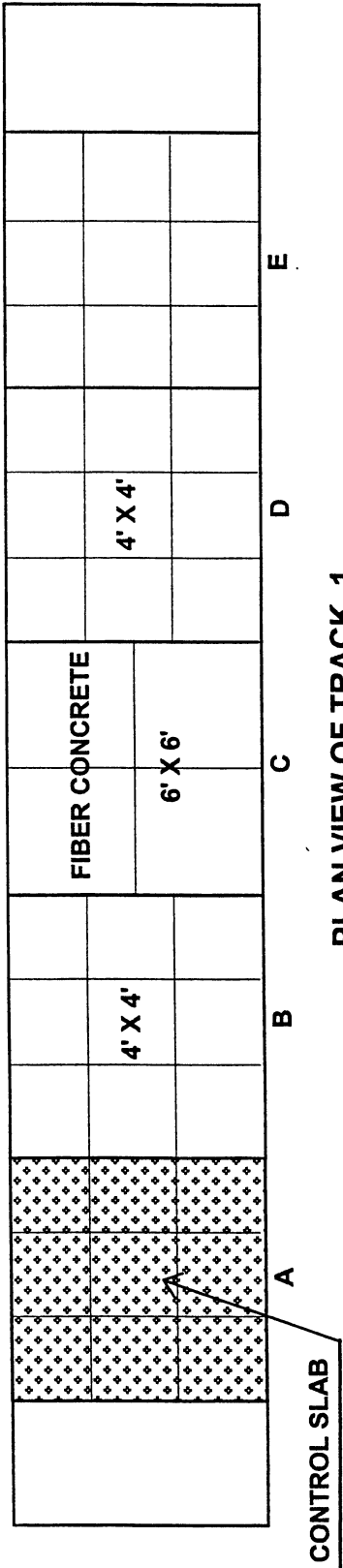
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4	FWD Deflection Basins on Asphalt and UTW Surfaces

Figure 1 ULTRA-THIN CONCRETE OVERLAY TEST TRACK 1

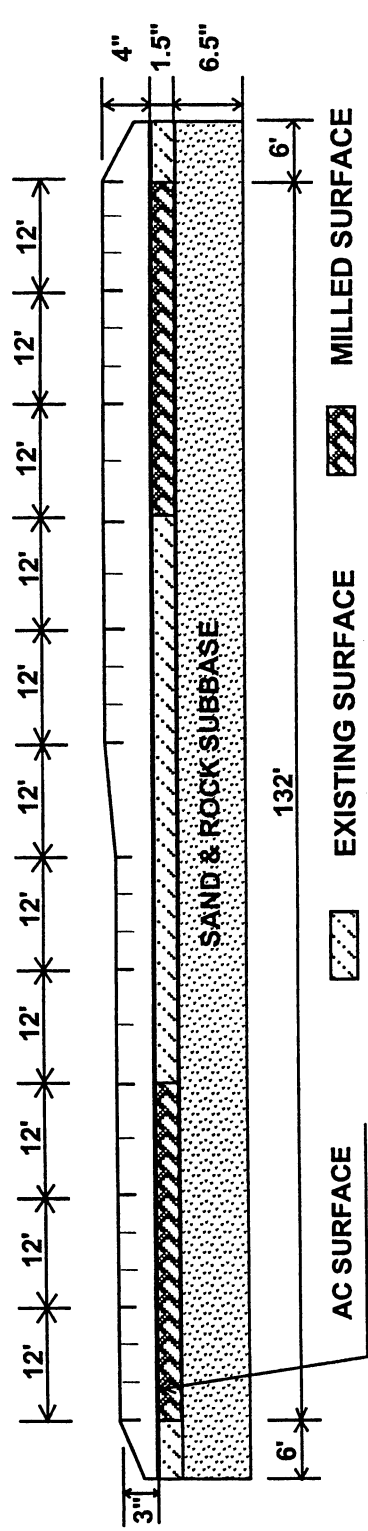


SIDE VIEW OF TRACK 1

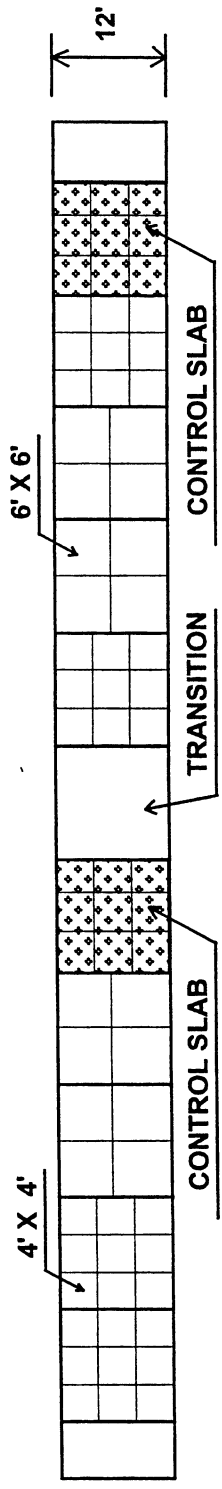


Note: Except control slabs, all others slabs include polypropylene fibers.

Figure 2 ULTRA-THIN CONCRETE OVERLAY TEST TRACK 2



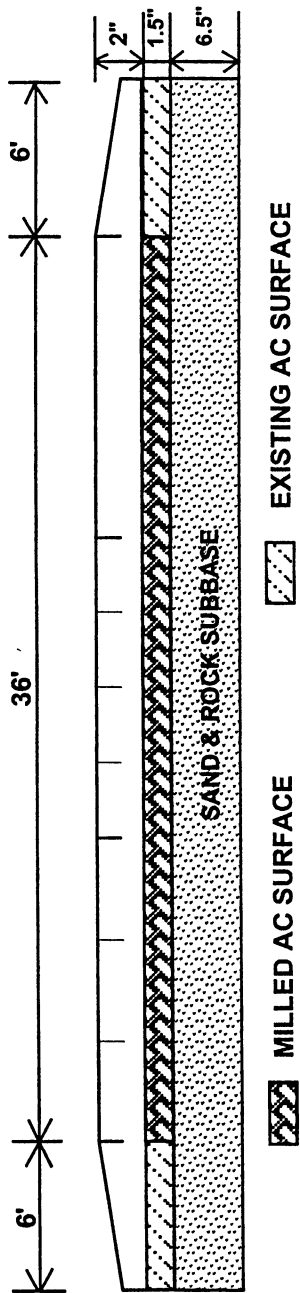
SIDE VIEW OF TRACK 2



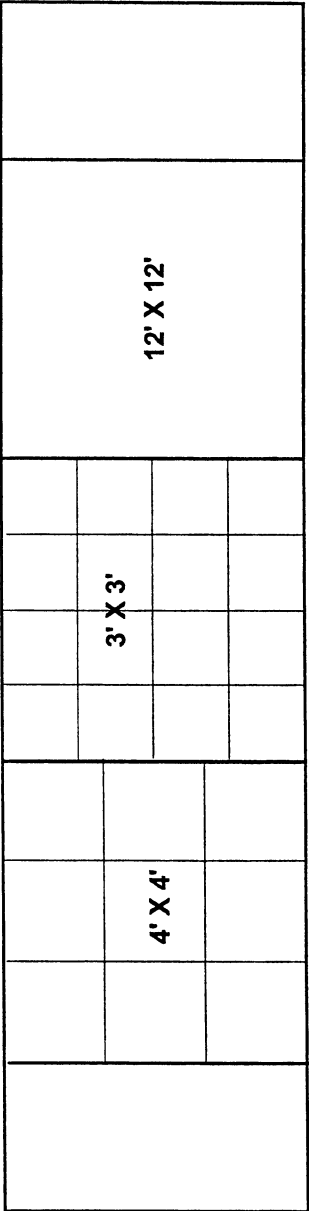
PLAN VIEW OF TRACK 2

Note: Except control slabs, all others slabs include polypropylene fibers.

Figure 3 ULTRA-THIN CONCRETE OVERLAY TEST TRACK 3



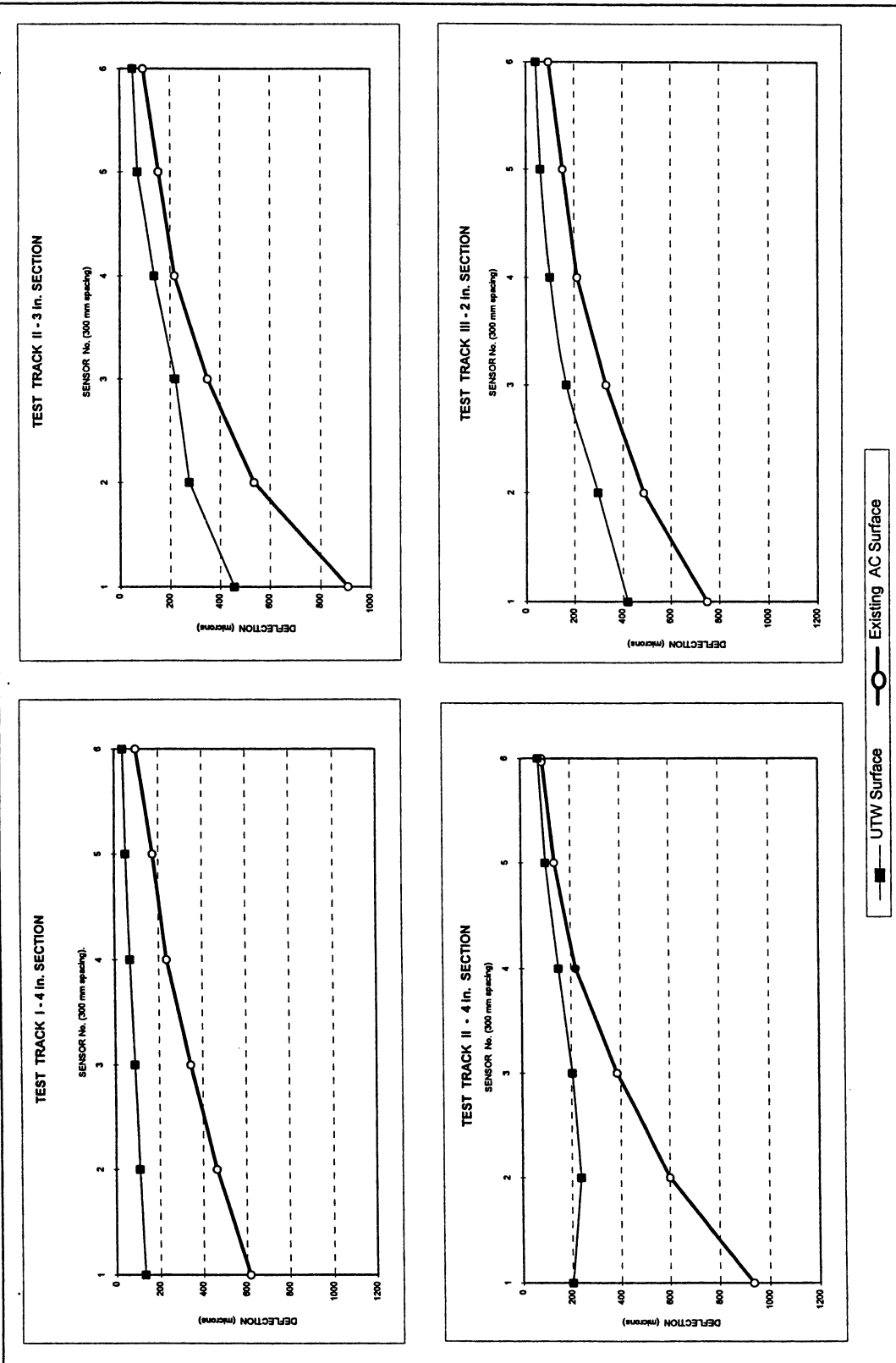
SIDE VIEW OF TRACK 3



PLAN VIEW OF TRACK 3

Note: All slabs include polyolefin fibers.

Figure 4 FWD Deflection Basins on Asphalt and UTW Surfaces



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ACKNOWLEDGMENTS

The authors wish to thank H. Godwin, J. Moxley, A. VanDyck and A. Malpartida for their significant contribution during the project planning and testing. The contribution of the engineers of the Pavement Systems Evaluation and the Structures Section of the State Materials Office (FDOT), Advanced Site & Paving, Florida Mining & Materials, Rinker, Florida Rock Industries, Florida Concrete and Products Association, Forta Corporation, 3M, Fibermesh, and FDOT's Gainesville Maintenance is much appreciated.