PERFORMANCE AND REPAIR OF UTW PAVEMENTS

Chung-Lung Wu, Ph.D., P.E.
Principal Engineer
Email: clwu@ctlgroup.com

and

Shiraz Tayabji, Ph.D., P.E.
Regional Manager
Email: stayabji@ctlgroup.com
Construction Technology Laboratories, Inc.
5565 Sterrett Place, Suite 312
Columbia, Maryland 21044
Telephone: (410) 997-0400; Fax: (410) 997-8480

Matthew Sheehan, P.E.
Engineer
Email: msheehan@ctlgroup.com
Construction Technology Laboratories, Inc.
5420 Old Orchard Road
Skokie, IL 60077
Telephone: (847) 965-7500; Fax (847) 965-6541

and

James Sherwood, P.E.
Highway Research Engineer
Email: jim.sherwood@fhwa.dot.gov
Federal Highway Administration
6300 Georgetown Pike
McLean, VA 22101
Telephone: (202) 493-3150; Fax (202) 493-3086

ABSTRACT

Since its inception in the early 1990s, ultra-thin whitetopping (UTW) technique has rapidly developed into a viable pavement rehabilitation alternative for deteriorated asphalt pavement. The development of the mechanistic design procedure in 1997 represented a milestone in the development of this technology. Over 150 UTW pavements have been built in the last decade.

As the UTW technology matures and with many existing UTW projects in services, it is essential to review the performance of existing UTW pavements to understand their behavior under traffic and to establish repair and rehabilitation techniques once distresses occur. The purpose of this

paper is to present the results of studies conducted dealing with UTW pavement performance reviews and repair and rehabilitation experiences. The effectiveness of using currently available concrete pavement repair and rehabilitation procedures is explored. The paper includes detailed performance descriptions of several existing UTW pavements in Georgia and Tennessee and the UTW repair and rehabilitation projects conducted at the Federal Highway Administration's (FHWA) Pavement Testing Facility and in several states.

INTRODUCTION

The ultra-thin whitetopping (UTW) technique involves placing very thin concrete slabs (51 to 102 mm) on old asphalt pavement to form bonded (or partially bonded) composite pavements. The reduction of thickness is justified by the use of high quality concrete with relatively high strength, shorter joint spacing, and a bond between the concrete and existing asphalt pavement.

The first UTW experimental project was constructed on the access road to a waste disposal landfill in Louisville, Kentucky in September 1991.[1,2,3] In this test section, two concrete slab thicknesses (51 and 89 mm) and two joint spacings (0.6 and 1.8 m) were used. The Louisville UTW pavement has performed well, carrying many more traffic loads than predicted by design procedures available at that time. Following the success of the Louisville UTW project, many other states, including Tennessee, Georgia, North Carolina, Kansas, Iowa, Pennsylvania, New Jersey, Colorado, Missouri, Mississippi, Virginia and Florida,[2,4-13] have constructed and are currently evaluating UTW projects. Over 150 UTW pavements have been built in the last decade. The development of a mechanistic design procedure for UTW pavements in 1997 represented another major step in the advancement of this promising technique.[14-16]

With the rapid development of the UTW technology and the many UTW pavements in service, it is important to review and document the performance of the existing UTW pavements under traffic, and to understand what factors would affect their performance. Also, with many of the early UTW projects nearing their design life, the next question that needs to be answered is how to repair the system when distresses do occur. Are the repair and rehabilitation methods used for conventional concrete pavements adequate for UTW pavements?

This paper presents the results of studies conducted to survey existing UTW pavement performance and to experiment with the repair and rehabilitation of some UTW test sections.

PERFORMANCE EVALUATION OF UTW IN GEORGIA AND TENNESSEE

Distress surveys were performed on several UTW sections in Georgia and Tennessee in 1995, using the Distress Identification Manual developed under the Long Term Pavement Performance (LTPP) Program.[17] Visually evaluated parameters included mapping crack locations, crack width, joint spacing, joint/crack faulting, surface wear, and approach and leave edge condition in relation to the surrounding asphalt pavement. Cores were removed to measure concrete and asphalt thickness and quality. When possible, a traffic count was performed to document the service the pavement provides. The projects studied are presented in Table 1.

Table 1. Ultrathin Whitetopping Project Field Evaluations

State	Location	Date Constructed	PCC Thickness, mm	Joint Spacing, m
Georgia	Lavonia – Georgia DOT I-85 Truck Scales	May 1993	64	0.6 by 0.6
Tennessee	Chattanooga – S. Hickory & E. 28th St.	Jul 1992	76	1.1 by 1.3
Tennessee	Chattanooga – Belvoir & Brainerd	Nov 1993	76	1.2 by 1.2
Tennessee	Nashville – N. 1st St. & Woodland Ave.	May 1992	64	1.4 by 1.4
Tennessee	Nashville – Deaderick between 4th & 5th	Oct 1994	102+	1.2 by 1.2

The results obtained from the survey of these sites are discussed individually in the following sections. General site observations are summarized and explanations for the performance of each section are presented based on the field surveys and laboratory evaluations. The observed UTW pavement performance was also compared with the performance predicted by the mechanistic design procedure developed in 1997.[14-16]

Lavonia, Georgia

This UTW site was located at a truck weigh station on the southbound side of Interstate 85 in Lavonia. Approximately 100 five-axle vehicles per day were reported to traffic the whitetopping sections to use the static load scales. As of May 1995, just prior to the evaluation, the Georgia DOT reported in their two-year follow-up inspection that the sections had experienced an estimated 351,000 80-kN Equivalent Single Axle Loads (ESALS).[18]

Design and Construction – Two sections of whitetopping have been monitored at this site. Approximately 43 m of the asphalt lane approaching the scales was milled to a depth of 64 mm and inlaid with polypropelene fiber reinforced concrete. On the exiting side of the scales, a 12-m section of asphalt was milled 64 mm deep and a non-fiber reinforced concrete inlay was placed. The fibrous and non-fibrous concrete had air contents of 6.3 and 2.8%, 24-hour compressive strengths of 26.3 and 30.1 MPa, and 28-day compressive strengths of 42.6 and 49.4 MPa, respectively. Transverse joints were first installed at 3.65-m intervals with one longitudinal sawcut located in the approximate center of the sections. Once the initial joints were cut, a lightweight dry cut saw was used to install joints on the 0.6-m grid system. The joints were cut 19 mm deep, and the pavement was opened to truck traffic after six days.[19]

Visual Survey – The surface of the whitetopping sections was given a coarse finish, which appears to be the source of the surface roughness observed rather than wear or erosion. Some grooves in the texture may have even led to initial cracking by creating a weakened plane. Cracking for both sections was recorded during the site inspection. Of the 537 0.6-m-square-slabs in the fibrous section, 10 (2%) were cracked. The cracks ranged in width from 0.33 to 1.5 mm as measured on the surface with an optical crack comparator. For the non-fibrous section, 6 of the 133 slabs (5%) exhibited cracks. One crack, affecting 3 slabs, appeared to be the result of late sawing or an inadequate joint depth. The remaining cracks in the non-fibrous section exhibited slight spalling. These slabs were in the right side wheelpath as trucks leave the static load scales and they also exhibited the most severe faulting found in either test section. Although SHRP P-338 does not offer severity levels for faulted joints, the maximum measured fault of 3.9 mm is not generally considered to be a problem.

Cores – Cores were obtained from each section to record the layer thicknesses. The core taken from the non-fibrous section measured 79 mm of concrete over 283-mm asphalt, and the core taken from the fibrous section revealed a similar 73 mm concrete over 276 mm asphalt. The cores were taken in the middle of the sections. The Georgia DOT reported that the thickness was increased to about 102 mm for approximately 1.8 m on the approach and leave ends of both sections due to load transfer concerns at the ends of the sections.

Bond – A direct shear strength test was performed on the core removed from the fibrous section. The nominally 95-mm-diameter core failed at a direct shear load of 2,534 N resulting in a shear strength of 0.35 MPa. The core removed from the non-fibrous section could not be tested for direct shear due to a horizontal crack within the asphalt layer resulting in an insufficient thickness of asphalt to perform the test.

Performance Evaluation – The general condition of the Georgia UTW site is presented in Figure 1. Both the fibrous and non-fibrous sections are performing very well. Overall, the fibrous section is exhibiting less deterioration than the non-fibrous section. Factors which contribute to its success include small joint spacing, sufficient joint sawcut depth and timing, adequate asphalt base, and proper asphalt preparation by milling.

With the observed pavement parameters and material properties, the UTW mechanistic design procedure developed in 1997 was used to predict the pavement performance. It was reported that the pavements had been subjected to 351,000 18-kip ESALs in two years after opening to traffic. As mentioned in the previous sections, the UTW pavement had concrete slab thickness of 64 mm, panel size of 0.6 by 0.6 m, and asphalt layer thickness of 276 mm. Using these parameters, it was estimated that, after two years of service, about 10% of the pavement's service life had been consumed, and the UTW pavement would have a service life of 20 years.

Chattanooga, Tennessee

The first whitetopping site constructed in Chattanooga was placed during July 1992 as a left turn lane from southbound S. Hickory to E. 28th Street. The section is located in a light residential/light industrial area of southwest Chattanooga.

Design and Construction – The whitetopping pavement section is approximately 40-m long and 4.6-m wide. The section is divided into 31 slabs longitudinally, each approximately 1.3-m-long, and 4 slabs transversely, each about 1.1-m-wide. The asphalt in this section was milled to a depth of approximately 76-mm, and a concrete inlay was placed in the milled region.

Traffic Survey – A traffic count was performed at three periods during the day to ascertain approximate traffic volumes the section was carrying. The vehicles observed were broken into three categories including passenger cars, busses, and trucks. Passenger cars include light automobiles and pick-up trucks with four-wheel loads. Busses include box-type delivery vans and school busses with less than ten wheels. Trucks include semi-tractor trailer vehicles with ten or more wheel loadings. During the survey date in

November 1995, the average number of vehicles per hour for the time surveyed included 160 cars, 10 busses, and 4 trucks.

Visual Survey – This particular UTW section was heavily damaged. Of the original 124 slabs, 22 (18%) had been replaced with new concrete slabs that exhibited little or no problems at the time of inspection. At least 62 (61%) of the remaining slabs were shattered and another 7 (7%) exhibited cracks of high-severity level. Permanent deformation under repeated traffic forced the City of Chattanooga to place a skim coat of asphalt over approximately 11% of the whitetopping section. In addition, a light rain the previous night left the section damp and pumping was noticed in the morning hours.

Cores – Two cores were removed from the UTW section. One core was drilled in a slab exhibiting minimal cracking, and a second core was drilled from a shattered slab. The first core consisted of 67 mm of concrete over 279 mm of asphalt. The second core from a damaged area measured 76 mm of concrete over a loose, possibly only asphalt treated base material. The asphalt from both samples was too damaged to allow shear testing of the bond interface. The concrete-asphalt bond failed during coring of the first sample and the asphalt fell apart while attempting to measure the sample density, and it was not possible to retrieve an intact asphalt sample at the second core location.

Performance Evaluation – This UTW section exhibited severe cracking at the time of the survey. Although the section was still functioning, its performance does not compare to other projects surveyed. The primary reason for the low level of performance is poor base material and/or poor bonding between the concrete and asphalt layers (Figure 2).

A UTW pavement design evaluation was also performed. The traffic data obtained during the survey was used as a basis to estimate the Average Daily Truck Traffic (ADTT) and the axle load distribution Category 2, as presented in Table 4 of the Portland Cement Association (PCA) Publication IS184.02P,[24] was assumed in the analysis. With a concrete slab thickness of 76 mm, joint spacing of 1.3 m and no asphalt layer, the UTW pavement would start cracking within the first year. The design of this UTW pavement was apparently inadequate (because of the extremely poor condition of the asphalt layer). Again, assuming that no asphalt layer exists, for a 12-year design service life, a pavement with a concrete slab thickness of 140 mm and joint spacing of 1.2 m would be required.

Another site near Belvior and Brainerd Road was also inspected during the time of this performance survey. The section is 46-m long, 3.7-m wide, and 76-mm thick with joints installed at approximately 0.9- and 1.2-m spacings. Due to heavy traffic volumes, core samples were not collected at this site. A visual evaluation of the slab conditions was performed. The section was constructed on a downward slope approach to an intersection that had exhibited rutting and shoving distresses while paved with asphalt. The distress observed in this UTW section appeared to be minimal; four corner cracks, one transverse crack, and a longitudinal crack extending through three slabs were observed (less than 4% of the slabs exhibited cracking). A slight loss of surface texture in certain areas, light joint spalling, and a fair degree of joint raveling likely due to early joint sawing operations were also observed. Overall the UTW section was performing very well.

Nashville, Tennessee

Two whitetopping sites in Nashville were evaluated for their performance. The first was located at 1st Street and Woodland Avenue, and the second was located at Deaderick and 4th Street.

1st Street & Woodland Avenue

Design and Construction – According to construction records, the concrete for this section was placed over an old cobblestone section. The cobblestone had been paved with a hot mix asphalt surface course, but milling operations performed in preparation for the whitetopping extended through the asphalt to the cobblestone layer in many areas. Areas where the asphalt remained were likely very thin especially towards the center or crown of the original roadway. The test section measures approximately 30 m long by 8.5 m wide. Joints were sawn at 1.4 m in the longitudinal direction and 1.4 m in the transverse direction. At the time of the inspection the section had reportedly serviced over 4 million ESALS since construction.[4]

Visual Survey – The pavement, which was to service a high amount of semi-tractor trailer vehicles exiting a fuel transfer depot, is exhibiting severe cracking and deformation similar to that seen in many rutted asphalt pavements. Cracking is concentrated in the vehicle wheelpaths and spreads due to traffic wander near the intersection. Even though there is severe cracking, the broken pieces of concrete remain in place and have not developed into potholes. The traffic sensors located beneath the concrete continue to function properly and the pavement remains part of a serviceable intersection.

Cores – Due to the volume of truck traffic leaving the transfer facility, core removal was limited to the edges of the pavement near the curb. In this area, the concrete was in relatively good condition as compared to the heavily trafficked, shattered interior slabs. The 76-mm concrete pavement was placed on approximately a 102-mm asphalt layer. This asphalt may only exist on the outer edges of the roadway and may have been previously used as a shoulder material.

Bond – Direct shear testing was performed on the bond between asphalt and concrete. The 95-m-diameter core resulted in 2,400 N of shear or 0.34 MPa shear strength. The shear failure occurred within the asphalt layer.

Performance Evaluation – Based upon the visual inspection and materials testing performed on the core, the cracking of this whitetopping section can be attributed to an insufficient asphalt section. According to several reports, the 76-mm concrete section was essentially placed directly on a cobblestone base, which is not adequate to carry the experienced heavy traffic loading. The core removed by CTL from the edge of the section near the sidewalk exhibited adequate interface shear strength as well as exceptional asphalt properties in comparison to previous projects studied. In conjunction

with the traffic loading, the relatively long 1.4-m joint spacing may have also contributed to the deterioration observed.

Again, the UTW pavement design was analyzed using the mechanistic design procedure. It was reported that the pavement had been subjected to four million ESALs in four years. From cores taken from the field, the pavement had a concrete slab thickness of 76 mm, with joint spacing of 1.4 m, and an asphalt layer of 100 mm. From the design evaluation, the UTW pavement appeared to be inadequate and the pavement would start cracking within the first year under such heavy truck traffic. A further analysis revealed that a concrete slab thickness of 203 mm would be required. Apparently, an UTW pavement would not be appropriate for pavements subjected to such heavy truck traffic.

Deaderick & 4th Street

Design and Construction – The Deaderick Avenue UTW section was constructed in a Bus Stop lane in downtown Nashville. The existing asphalt was milled to at least a 102-mm depth, a fiber reinforced concrete inlay was placed in the 101- by 3.7-m lane, and joint spacings were 1.2 m in both directions. The section is reportedly trafficked by 475,000 busses annually,[4] which resulted in severe rutting when the section was constructed with asphalt.

Visual Survey – Of the 500 slabs surveyed, only 5 (1%) exhibited cracks and none of the cracks showed signs of excessive movement or wear due to traffic. Three of the cracked slabs are located over a steam line that reportedly caused the crack shortly after construction. Even areas that have irregular joint layouts, i.e. around drainage inlets and access boxes, show little sign of re-entrant cracking as might be typically expected; only one slab was observed to have a re-entrant crack.

Cores – Two cores were removed from the whitetopping section. The cores were measured to consist of 152 and 140 mm of concrete over 121 and 102 mm of asphalt for the first and second core, respectively. Direct shear strengths were measured to be 0.42 and 0.82 MPa for cores one and two, respectively. Due to the relatively thick concrete layer negating this project as a true ultra-thin whitetopping section, additional asphalt testing was not performed.

Performance Evaluation – This section exhibited minimal cracking. The relative success of this section may be attributed to the relatively high thickness of concrete used, the adequate asphalt layer, and the sufficient bond strength. Pavement design evaluation conducted using the mechanistic design procedure further confirmed the adequate design of this UTW pavement. The annual bus traffic was reported to be 475,000. With a 152-mm concrete slab, with joint spacing of 1.2 m, and an asphalt layer of 100 mm, the design service life would be 12 years for this UTW pavement.

REPAIR OF UTW PAVEMENTS

As UTW technology evolves, many early projects have reached their design lives and some have started showing signs of distresses. Therefore, the next step in UTW technology development would be the development of repair and rehabilitation methods. Are repair methods used for conventional concrete pavements applicable for UTW pavements? This part of the paper presents some repairs that have been used in New Jersey and Iowa and an experimental repair project conducted at the FHWA's Turner-Fairbank Highway Research Center (TFHRC).

New Jersey Experience

The New Jersey Department of Transportation (NJDOT) constructed a UTW pavement test section on a ramp connecting I-295 northbound and Route 130 northbound in 1994.[20] The UTW test section consisted of three panel dimensions (0.9, 1.2, and 1.8 m), with a total project length of about 275 m. Because of geometric limitations, the test pavements were constructed in two 3.7-m wide segments, with a construction joint between them. Data obtained from cores showed an average concrete overlay thickness of 97 mm, ranging from 74 to 117 mm. Average existing asphalt layer thickness (after milling) was 168 mm, ranging from 132 to 188 mm.

The NJDOT performed an evaluation of the UTW test section in 1997. It was reported that panels with 0.9-m dimensions performed the best while panels with 1.8-m dimensions showed the most severe distress. Discussions with NJDOT personnel indicated that most of the distresses occurred along the longitudinal construction joints, which were subjected to frequent truck crossing. Load transfer across transverse joints was over 80% for most joints.

In September 1998, NJDOT identified areas with severe distress that needed corrective work (about 466 m^2 total).[21] In most cases, the center section of the pavement was identified as needing replacement. The concrete used in the repair was specified to have a compressive strength of 21 MPa in 12 hours. The panels with 1.8 by 1.8-m dimension originally were reduced to 0.9 by 0.9 m. Joints were sawed to 25-mm deep by 3.2-mm wide. The concrete was specified to be wet-cured until it reached the required strength and then further cured with a white pigmented curing compound.

The repair work was carried out in November 1998. The concrete overlay removal and replacement consisted of the following steps:

A. Marking repair boundaries and sawing concrete overlays full depth.
B. Milling most of the concrete.
C. Using a backhoe and jackhammer to remove the remaining concrete (adjacent to concrete not to be replaced).
D. Examining the asphalt layer. The asphalt surface was in good condition.
E. Air-blasting to clean the asphalt surface before concrete placement. No treatment or preparation was needed on vertical joint surfaces along repair boundaries.
F. Placing concrete following normal NJDOT practice.
G. Sawing joints (early entry sawing did not work because of fast strength gain).
H. Curing concrete by wet cure followed by a white pigmented curing compound.

The entire repair work was completed in 12 hours.

Iowa Experience

In 1994, the Iowa Department of Transportation (Iowa DOT), in conjunction with FHWA and Iowa State University, constructed an 11.7-km UTW research project. This project is located on Iowa Highway 21 in Iowa County and consisted of 41 UTW test sections. The major variables included in the construction of the test sections were slab thickness, panel dimension, asphalt pavement surface preparation, and the use of fibers in the concrete. A weigh-in-motion device was installed in each lane near the center of the project to monitor traffic. In 1997, an average of 40 ESALs per day were recorded northbound and 20 ESALs per day southbound.[8] All test sections have performed well to date with only minor surface distresses being noted after three years of service, with less than 2% of the slabs showing visual distresses.[9]

In August 1999, the research project was extended for an additional five years, providing opportunities for experimentation with potential UTW pavement rehabilitation and repair techniques. The new phase of this project started with the removal and replacement of the cracked panels in the 51-mm-thick UTW sections.[10] Twenty-nine areas, with variable sizes, were identified as needing rehabilitation and repair. The criteria used in the selection of the repair areas were (1) individual panels separated into four or more pieces by cracks and (2) areas exhibiting characteristics of potential debonding.

The UTW panel removal and replacement were conducted between August 4 and August 20, 1999 and consisted of the following steps:

A. Identifying repair boundaries (generally matched the existing joints in the UTW panels).
B. Sawing the concrete to a depth of approximately 70-mm along the repair boundaries.
C. Removing distressed panels.
D. Cleaning the repair areas and roughening the AC surfaces.
E. Placing concrete (or asphalt in some areas) following normal practice.
F. Curing concrete using white pigmented curing compound.

Different techniques were used for UTW panel removal, depending on the number of panels and sizes of the repair areas, and their orientation along the cross section of the highway. A backhoe was the most used equipment for UTW panel removal, especially on larger removal areas. A small front-end loader was also used to pry out strips of panels that were not wide enough for backhoe operation. The small front-end loader appeared to be effective in removing the panels since the UTW panels were observed to be essentially de-bonded from the underlying AC layers.

In some instances, all four sides of the panels were not exposed to allow the access of the backhoe or the front-end loader. These panels were pulled out by drilling two holes into the pavement and inserting metal rods connected to the backhoe by chains. The thickness of the repaired panels was about 25 mm thicker than that of the original UTW panels. No signs of distress have been observed for the repaired panels to date.

UTW Repair Demonstration at the FHWA's UTW Test Sites

Background

In April 1998, under a cooperative research agreement with FHWA, the American Concrete Pavement Association (ACPA) constructed eight full-scale UTW test pavements.[22] The test pavements are located at FHWA's Pavement Testing Facility in McLean, Virginia. The test UTW pavements consist of eight lanes (designated as Lanes 5 to 12) with different combinations of design variables. The design variables included two levels of remaining asphalt layer thickness after milling (114 and 140 mm), two levels of concrete overlay thickness (64 and 89 mm), three levels of joint spacing (0.9, 1.2, and 1.8 m), and the use of fibers.

Load testing was conducted between May 1998 and November 1999, using FHWA's Accelerated Loading Facility (ALF). Detailed distress surveys were performed on a regular basis at each test section. Some of the sections exhibited cracking under the heavy loading according to the design. Major forms of distresses observed were corner breaks, transverse cracking, and longitudinal cracking. Faulting along longitudinal and transverse joints were also recorded.

During early 2000, it was decided to further load test some of the test sections during the spring of 2000 in an attempt to develop additional distresses in the test sections. This offered an opportunity to experiment and demonstrate some UTW repair methods. A work plan was formulated and proposed in March 2000 to repair the distressed panels on the Lane 6 and Lane 10 test sections. The test sections would then be subjected to additional ALF load testing to also evaluate the performance of the repaired test sections.

General Conditions of Selected UTW Pavement Test Sections

Slab panels selected from Lanes 6 and 10 test sections were used to demonstrate the panel removal and replacement. Lane 6 pavement consisted of a nominal design concrete overlay of 64 mm over a 140-mm asphalt layer, with 1.2-m joint spacing. Lane 10 was designed to have a nominal concrete overlay of 89 mm on top of a 114-mm thick asphalt layer, with 1.8-m joint spacing. Neither section used fiber in the concrete mix. A summary of the testing status and current conditions of the selected UTW sections is presented in Table 2. A general layout of the two pavement sections and the numbering of the individual slabs are shown in Figure 1. The most up-to-date distresses before repair are also presented in Figure 3 for these two UTW test sections.

Table 2. Testing Status and Current Conditions for Lane 6 and Lane 10 UTW Sections

Lane No.	Actual No. of Load Passes	Current Pavement Conditions			
		No. of Cracked Slabs	Type of Major Cracking	Longitudinal Joint Faulting, mm	Transverse Joint Faulting, mm
6	359,000	11	Corner Breaks Longitudinal Cracking	From 2 to 8 Average 2 mm	No measurable faulting
10	441,000	4	Corner Breaks Transverse Cracking	From 2 to 5 mm Average 2 mm	From 0 to 4 mm

Referring to Figure 3, Panes 7B, 7C, 10B, 11B, and 11C of Lane 6 and Panels 3A, 6A, and 6B of Lane 10 were selected for panel removal and replacement. In addition, an optional step was also included for removing and replacing Panels 4A to 8A of Lane 9 (same design as the Lane 10 section). The purpose of this optional plan was to demonstrate the use of a front-end loader for removing multiple UTW panels.

Guidelines for UTW Panel Removal and Replacement

Guidelines for UTW panel removal and replacement used in this study were established based on work conducted by New Jersey, Iowa, and other states. The guidelines consisted of the following:

A. Identify and mark the distressed panels to be removed. Only those panels that appear to have dislodged or moved were targeted for repair.
B. Mark boundaries of sawcut along exterior joints. The lines of sawcut would be 152 mm inward from joints.
C. Sawcut the panels full depth using a saw with a diamond blade.
D. Remove the isolated slabs by:
   - Breaking up the interior concrete using a 13.6-kg maximum pneumatic hammer and removing the broken concrete pieces.
   - Removing the remaining 152-mm wide concrete around the perimeter using a light pneumatic hammer (6.8 kg is recommended, 13.6 kg maximum). This will create rough joint surfaces for better aggregate interlock.
E. For multiple-slab removal (optional):
   - Removing the first slab following the procedure described in Items A, B, and C.
   - Using a small front-end loader to lift out the remaining panels. Take care not to damage adjacent panels. Note: It may be necessary to sawcut full depth along the periphery of the two longitudinal joints to facilitate the slab removal operation.
F. Prepare repair area by:
   - Removing any loose materials (concrete as well as asphalt concrete).
   - Cleaning the asphalt concrete surface thoroughly by air blasting.
G. Place new concrete:
   - Placing concrete over the prepared asphalt surface. The concrete was specified to have a compressive strength of 21 MPa at 24 hours or 28 MPa at 72 hours.
   - Consolidating concrete using a handheld vibrator.
   - Finishing concrete using a straight edge or vibratory screed to meet the existing surface profile.
   - Texturing the concrete surface (burlap drag) to match the surrounding pavement slabs.
   - Applying membrane-forming curing compound to the slab surfaces at a maximum coverage rate of 2.46 m^2/L. The curing compound shall be applied immediately after the final surface texture has been provided and water sheen has disappeared.
   - Covering the newly placed concrete with an insulating blanket if the concrete or ambient temperature falls below 15.6°C within eight hours of concrete placement.

- Sawcut joints to match existing joints, if needed. Joint depth shall be 1/3 of the slab thickness, and joint width shall be 3.2 mm. Sawing shall start as soon as the surface will permit it, without causing raveling.

Summary of Field Operations

The field operations of panel removal and replacement followed essentially the repair guidelines outlined in the previous section (Figure 4). Time required for conducting each step was recorded that could be used as reference for future repair work of UTW. Please refer to Reference 23 for detailed descriptions of the field operations and other project related data. The following items are some general observations made from the field operations:

- Although the UTW panels have cracked, the bond between the concrete and the asphalt layers remained very strong. Some asphalt came out with the concrete removed, and the final replaced panel depth was about 25 mm thicker than the original panels.
- Because of the strong bond, the front-end loader was unable to remove multiple panels.
- UTW panel removal was started from slab interior and proceeded outward to avoid damaging the surrounding concrete.
- Time required for UTW repair was one hour for the 1.8 by 1.8 m panels (Lane 10) and 45 minutes for the 1.2 by 1.2 m panels (Lane 6). About half of the time required was for panel removal.
- Field QA/QC tests were conducted and the test results are presented below:

Air Temperature	17.4°C at 11:20 am, April 20, 2000
Average Concrete Temperature	23.2°C, measured at 11:25 am, April 20, 2000
Concrete Slump	25.4 mm
Unit Weight	2,332 kg per cubic meter
Plastic Air Content	4.5%

- Compressive strength, flexural strength (third point), splitting tensile strength, and modulus of elasticity were measured at 24 hours, 7 days, and 28 days. Summary statistics are presented in Table 3.

Table 3. Summary of Strength Test Results

Test	Age, days	Average, MPa	Standard Dev., MPa	Coefficient of Variation, %	No. of Samples
Compressive	1	18.0	0.3	1.4	4
	7	28.1	1.2	4.2	4
	28	36.1	1.6	4.5	4
Flexural	1	3.79	0.30	7.8	3
	7	4.72	0.12	2.5	4
	28	5.17	0.12	2.4	4
Splitting Tensile	1	2.69	0.08	3.1	4
	7	3.41	0.14	4.1	4
	28	3.91	0.25	6.5	4
Elastic Modulus	1	18,948	345	2.0	2
	7	26,527	1,102	4.2	2
	28	28,318	3,238	11.4	2

Performance of the Repaired UTW Panels

ALF load testing for the repaired Lanes 6 and 10 test sections started eight days after the repair. The ALF testing was performed using dual wheels with a total load of 53.4 kN. At the completion of the ALF loading tests, Lane 6 panels had been subjected to 400,000 load repetitions and Lane 10 to 427,000 load repetitions. The design load repetitions for the original UTW pavements were 255,000 and 441,000 for Lane 6 and Lane 10 test sections, respectively. Two of the five repaired Lane 6 panels and two of the three Lane 10 repaired panels exhibited cracks. Overall, there were five cracks on Lane 6 and two cracks on the Lane 10 repaired panels. A few joint and corner spalls were also observed. The following is the development of the cracks:

Lane 6 UTW Test Section

A corner crack on Panel 10B after 66,700 passes
A corner crack on Panel 7C after 105,100 passes
The second crack on Panel 7C after 182,000 passes, intersecting the previous corner crack.
A short crack inside the corner crack of Panel 10B after 316,000 passes
A short transverse crack at mid-panel (about 220 mm long) on Panel 10B after 375,000 passes

Lane 10 UTW Test Section

A corner crack on Panel 3A after 50,000 passes
A crack on Panel 6A after 400,000 passes

Most of the cracks were tightly held together. The repaired panels appear to have performed well. It is interesting to note that two cracks occurred on a panel that was not directly subjected to ALF loading (Panel 7C).

SUMMARY AND CONCLUSIONS

From field evaluation of UTW performance and repair practices, the following observations were made:

1. Good interface bonding between concrete and asphalt layers is essential for successful UTW pavement performance.
2. An adequate asphalt layer is required for good UTW performance. As suggested in the design guidelines developed by ACPA, a minimum asphalt thickness of 75 mm is recommended.
3. The developed ACPA design procedure can predict the UTW pavement performance adequately.
4. The most commonly used repair method for UTW pavements are panel removal and replacement, which has been demonstrated as an effective method for UTW repair.
5. Several different methods have been used for UTW panel removal, including lift-out method and the use of backhoes, front-end loaders, and jackhammers.

6. Guidelines for UTW panel removal and replacement has been proposed.
7. The repaired UTW panels have performed well under the accelerated load testing and has substantially extended their service lives.

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Figure 1. Lavonia, Georgia UTW Test Section



Figure 2. Chattanooga, Tennessee UTW Test Section

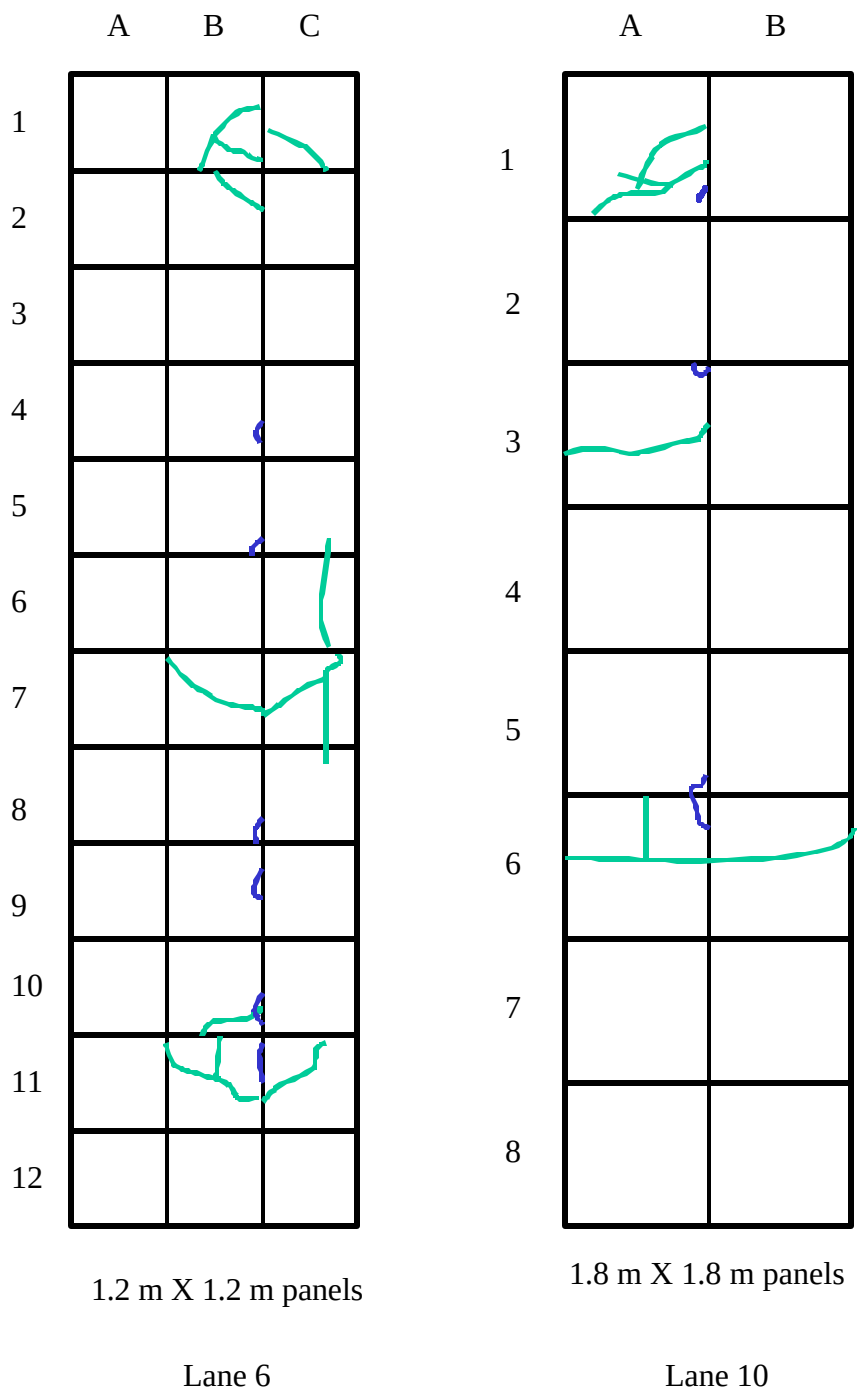

Figure 3. General Layout of the Two UTW Pavements and Their Associated Numbering and Distress



Sawing repair boundaries



Panel removal



Prepared repair areas



Concrete placement



Texturing



Applying curing compound

Figure 4. UTW Panel Removal and Replacement