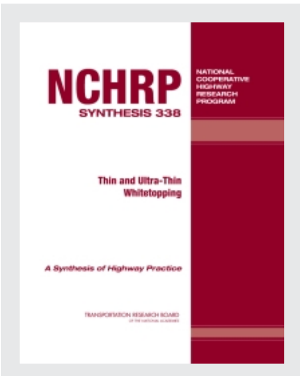

Thin and Ultra-Thin Whitetopping

DETAILS

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP SYNTHESIS 338

Thin and Ultra-Thin Whitetopping

A Synthesis of Highway Practice

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FOREWORD

By Staff
Transportation Research Board

Highway administrators, engineers, and researchers often face problems for which information already exists, either in documented form or as undocumented experience and practice. This information may be fragmented, scattered, and unevaluated. As a consequence, full knowledge of what has been learned about a problem may not be brought to bear on its solution. Costly research findings may go unused, valuable experience may be overlooked, and due consideration may not be given to recommended practices for solving or alleviating the problem.

There is information on nearly every subject of concern to highway administrators and engineers. Much of it derives from research or from the work of practitioners faced with problems in their day-to-day work. To provide a systematic means for assembling and evaluating such useful information and to make it available to the entire highway community, the American Association of State Highway and Transportation Officials—through the mechanism of the National Cooperative Highway Research Program—authorized the Transportation Research Board to undertake a continuing study. This study, NCHRP Project 20-5, "Synthesis of Information Related to Highway Problems," searches out and synthesizes useful knowledge from all available sources and prepares concise, documented reports on specific topics. Reports from this endeavor constitute an NCHRP report series, *Synthesis of Highway Practice*.

This synthesis series reports on current knowledge and practice, in a compact format, without the detailed directions usually found in handbooks or design manuals. Each report in the series provides a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems.

PREFACE

This report of the Transportation Research Board summarizes available information to document how departments of transportation and other agencies and owners are currently using thin and ultra-thin whitetopping (TWT and UTW) overlays among various pavement rehabilitation alternatives. This study covered all stages of the proper application of whitetopping overlays, including project selection, design, materials selection, construction, maintenance, and eventual rehabilitation or replacement. This synthesis provides the practitioner with a comprehensive source for the state of the practice, as well as the state of the art in TWT and UTW overlay application. It is designed to serve as a quick reference guide and as a training aid.

This synthesis report included a review of published literature related to all stages of the proper application of whitetopping overlays and a broad range of citations has been gathered, from practical case studies to reports of theoretical modeling. In addition, a survey of the highway community was undertaken that provided first-hand results of experiences with TWT and UTW. Responses were received from both the public and private sectors.

A panel of experts in the subject area guided the work of organizing and evaluating the collected data and reviewed the final synthesis report. A consultant was engaged to collect and synthesize the information and to write the report. Both the consultant and the members of the oversight panel are acknowledged on the title page. This synthesis is an immediately useful document that records the practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As progress in research and practice continues, new knowledge will be added to that now at hand.

CONTENTS

THIN AND ULTRA-THIN WHITETOPPING

SUMMARY

The purpose of this synthesis is to summarize available information and to document how departments of transportation and other agencies and owners are currently using thin whitetopping (TWT) and ultra-thin whitetopping (UTW) overlays among their various pavement rehabilitation alternatives. Although TWT and UTW overlays have been constructed for decades, their recent popularity is largely the result of a renewed demand for longer-lasting but cost-effective solutions for hot-mix asphalt (HMA) pavement rehabilitation.

A whitetopping overlay is constructed when a new portland cement concrete layer is placed on top of an existing HMA pavement system. The concrete thickness for a UTW is equal to or less than 100 mm (4 in.). A TWT is greater than 100 mm (4 in.) but less than 200 mm (8 in.). Conventional whitetopping is an overlay of 200 mm (8 in.) or more. In most cases, a bond between the new concrete and existing HMA layers is not only assumed during design, but specific measures are taken to ensure such a bond during construction. The success of this bond, leading to composite action, has been found to be critical to the successful performance of this pavement-resurfacing alternative.

The goal of this synthesis is to collect and report information about the use of TWT and UTW overlays within the highway community. Proper application of whitetopping overlays requires attention at all stages, including project selection, design, materials selection, construction, maintenance, and eventual rehabilitation or replacement. This synthesis explores the literature for topics related to each of these stages. A broad range of citations related to whitetopping has been gathered, from practical case studies to reports of theoretical modeling. In addition, the results of a survey of the highway community were used to provide firsthand insight of their experience with these overlays.

This synthesis provides the practitioner with a comprehensive source for the state of the practice as well as the state of the art in TWT and UWT overlay application. An approach to the proper application of this rehabilitation alternative is also presented, along with accessible citations of third-party information.

CHAPTER ONE

INTRODUCTION

BACKGROUND

Portland cement concrete (PCC) overlays over existing hot-mix asphalt (HMA) pavements have been used as a rehabilitation option for more than 80 years. Coined "whitetopping" by the industry, these overlays have been used on airports; Interstate, primary, and secondary highways; local roads and streets; and parking lots to improve the performance, durability, and riding quality of deteriorated HMA surfaces.

Modern whitetopping overlays are commonly classified by thickness and by bond with the HMA. Three distinct categories are found in the literature (*1,2*):

1. Conventional whitetopping—a concrete overlay of 200 mm (8 in.) or more, designed and constructed without consideration of a bond between the concrete and underlying HMA.
2. TWT—an overlay of greater than 100 mm (4 in.) and less than 200 mm (8 in.) in thickness. In most but not all cases, this overlay is designed and constructed with an intentional bond to the HMA.
3. UTW—with a thickness equal to or less than 100 mm (4 in.), this overlay requires a bond to the underlying HMA to perform well.

NCHRP Syntheses of Highway Practice 99 and 204, published in 1982 and 1994, respectively, document the historical use of whitetopping (*3,4*). These documents identify most whitetopping projects being constructed as using "jointed plain concrete pavement." Other types include "jointed reinforced concrete pavement" and "continuously reinforced concrete pavement." In the current synthesis, consideration is given only to whitetopping overlays constructed as jointed plain concrete pavement. Fiber-reinforced concrete (FRC) is commonly used with almost all UTW and some TWT overlays, and although considered a separate classification in previous NCHRP syntheses (*3,4*), in this document, it will instead be considered a characteristic of the overlay.

Although whitetopping overlays have been used in the United States since 1918, there has been renewed interest in them over the last 10 years as a result of several successful high-profile projects (*4–6*). In addition, responding to demand caused by rapidly deteriorating highways with only modest increases in funding, the concrete paving industry has adopted whitetopping as a key marketing strategy.

The survey conducted as part of this synthesis revealed that more than 50% of the states have constructed a TWT or UTW overlay within the last 5 years and more than 40% have built a project within the last year. In the 10-year period from 1992 to 2001, the American Concrete Pavement Association (ACPA) recorded 282 UTW projects in 35 states, totaling 765,000 m^2 (916,000 yd^2) (*7*).

TWT overlays are used on highway and secondary roads carrying a wide range of traffic, from light to heavy (*1*). General aviation airports have also used this class of whitetopping for runway, taxiway, and apron pavements (*8*). Because they are thinner, UTW overlays are best suited for more lightly loaded pavements. These pavements include some intersections, ramps, and light aircraft aprons (*9*). Overall, it appears from the information gathered that the use of TWT and UTW overlays is on the increase.

SYNTHESIS FOCUS AND OBJECTIVE

Although conventional whitetopping closely resembles most PCC pavements with respect to design, construction, and maintenance techniques, TWT and UTW possess some unique characteristics, most notably their thickness, joint spacing, and bond to the underlying HMA (*10*).

The goal of this synthesis is to collect and report information about the use of TWT and UTW overlays within the highway community. Proper application of these overlays requires attention at all stages, including project selection, design, materials selection, construction, maintenance, and eventual rehabilitation or replacement. During the development of this synthesis, the literature was reviewed for topics related to each of these stages. A broad range of citations related to TWT and UTW overlays have been identified, from practical case studies to reports of theoretical modeling. In addition, this synthesis presents the results of a survey of the highway community in regard to their firsthand experience with these overlays. This survey includes responses from both the public and private sectors.

The objective of this synthesis is to convey the information compiled in a manner that is easily accessible and understandable by the highway practitioner. Although a particular emphasis is given toward providing qualitative guidelines on TWT and UTW overlays, numerous references are cited that more fully document the analytical approaches used in understanding the nature of this rehabilitation technique.

SYNTHESIS ORGANIZATION

This synthesis report is organized into seven chapters: this introductory chapter (chapter one); an overview chapter (chapter two); a chapter on project selection (chapter three); one on whitetopping design (chapter four); one on construction practices (chapter five); a chapter on performance, repair, and rehabilitation (chapter six); and a summary chapter with conclusions and suggestions for further research (chapter seven). Supporting information follows, including the survey questionnaire and a summary of responses (Appendix A) and a select number of case studies on TWT and UTW projects (Appendix B). The user is encouraged to explore these case studies, because many of the concepts included in this synthesis were used on those projects.

The synthesis has been developed with two uses in mind. The first is to serve as a quick reference guide, each chapter having been developed in a hierarchical fashion, quickly allowing readers to identify their topics of interest. The second use is as a training aid, with a flow inherent in the material, allowing interested readers to gain a much better understanding of this topic as they read the entire report.

CHAPTER TWO

OVERVIEW AND APPLICATION

Whitetopping overlays have proven to be a successful pavement rehabilitation method. The intent of this report is to identify specific cases of their application, along with suggested practices to help ensure proper planning, design, construction, and maintenance. This synthesis will provide a summary of the state of the practice and a glimpse of the state of the art.

DEFINITIONS

For the purposes of this synthesis, the first term that requires definition is whitetopping: Whitetopping is a PCC layer constructed atop an existing HMA pavement structure (*2*). Whitetopping differs from other concrete overlay types, which include

- Bonded concrete overlay—a PCC layer constructed directly atop an existing PCC pavement. Specific construction techniques are employed to ensure a sound bond between the layers (*4*).
- Unbonded concrete overlay—a PCC layer constructed directly atop an existing PCC pavement but intentionally separated by a bond breaker, commonly consisting of HMA (*4*).

Figure 1 illustrates these overlay types.

Within the category of whitetopping, there are additional classifications of conventional whitetopping, TWT, and UTW. These terms were defined in chapter one.

One possible source of confusion in the definition of whitetopping pertains to the degree of bonding between the new PCC and existing HMA. Although TWT overlays have been reported in the literature dating back to 1918, little is noted about the specific construction techniques used until more recently. For example, no measures were reported as having been taken to enhance the bond between the PCC and HMA layers. To the contrary, previous recommendations for whitetopping have included application of a whitewash (e.g., lime slurry or curing compound) to assist in breaking the bond (*1*). Because conventional concrete pavement design techniques assume the underlying HMA to be a "stiff subbase," a lack of bond did not have a negative impact on the performance of the overlay (*11*).

In the mid-1980s, a new concept was advanced by researchers in Europe, where a bond between the PCC and the HMA would be considered in the design and intentionally introduced during construction by means of texturing of the HMA before the overlay (*12*). When a sound bond is assumed to exist, the concrete overlay can be of a thinner design, and thus UTW is possible.

In summary, it will be assumed in this report that UTW overlays should always be designed and constructed to maximize the bond between the PCC and HMA. The same assumption will usually be made for TWT; however, specific guidance will also be given for the design and construction of thicker TWT sections that do not necessarily assume bonding. Finally, although conventional whitetopping overlays are not considered in this synthesis, it should be noted that they are commonly designed and constructed as assuming little or no bonding action.

HOT-MIX ASPHALT PAVEMENT REHABILITATION

The behavior and performance of HMA pavements varies widely. Depending on the original design and ultimate use of the pavement structure, the HMA can ultimately fail in a number of ways, or the sources of failure may be combined. The most common failure modes of HMA pavements that can lead to the need for rehabilitation include the following:

- Rutting—pertains to permanent deformation in the wheelpaths caused by a combination of densification and shearing of the various pavement layers. The viscous properties of the asphalt binder (especially at high temperatures) are a significant factor leading to rutting in the HMA layer. Rutting in the underlying layers can be aggravated by the presence of moisture and high strains (*12*).
- Fatigue cracking—as with many engineering materials, repeated loading can damage an HMA, leading to the development of visible cracking. This failure mode is complex and can develop in a number of ways (*13*). It is commonly aggravated by a weak support system (e.g., owing to a saturated condition) and/or a stiffening of the HMA.
- Thermal cracking—at low temperatures, HMA responds in a more elastic and brittle fashion. If the pavement temperature lowers rapidly or frequently, transverse cracking can form on the pavement surface (*13*).

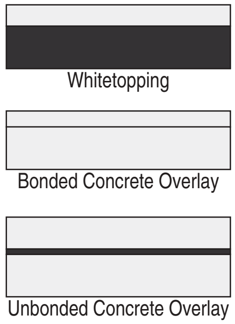

FIGURE 1 Concrete overlay types.

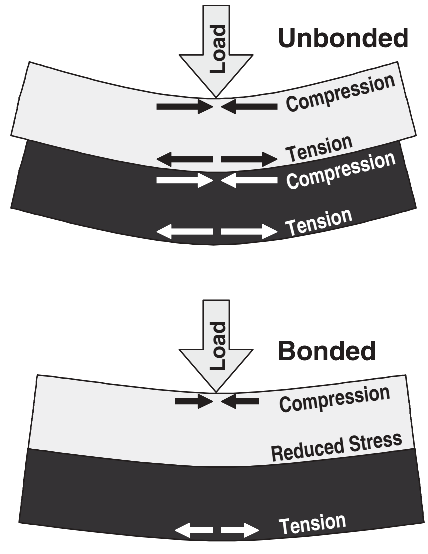

FIGURE 2 Effect of composite action on UTW and TWT behavior under flexural loading.

Other common structural and material failures include block cracking (the result of oxidation and shrinkage of the HMA), stripping (separation of the asphalt binder from the aggregate owing to the presence of moisture), and bleeding (excess asphalt binder forced from the matrix to the outside of the HMA layer).

In addition to undergoing structural and material failures, an HMA pavement may experience a loss of function. These failure modes include degradation of ride quality (pavement smoothness), loss of skid resistance, and an increase in noise.

By far the most common rehabilitation method for existing HMA pavements is an HMA overlay. Depending on a number of factors, the HMA overlay may be preceded by a milling operation (*14*). HMA overlays commonly "correct" functional failures, and depending on the particulars of the project, they can be used to address some structural failures (*15*).

Whitetopping overlays provide the industry with an alternative to HMA overlays. Both UTW and TWT are intended to correct structural and functional distress in an existing HMA pavement at a cost that is comparable to that for an HMA overlay, especially if life-cycle costs (LCC) are a consideration (*1*).

FUNDAMENTAL BEHAVIOR

UTW and TWT overlays provide a unique pavement structure that is fundamentally different from other pavement types. UTW and, in most cases, TWT overlays are designed and constructed with consideration of a sound bond between the PCC and HMA materials (*16*). The result is a composite structure that distributes traffic and environmental loading differently than more conventional PCC or HMA pavement structures.

As Figure 2 illustrates, the stress distribution in a bonded systems versus that of an unbonded system can be significantly different. As a result of the composite action, the stresses in the top (PCC) layer are significantly lower in the bonded than those in the unbonded case. Furthermore, because much of the slab is in compression, and because concrete is much stronger in compression than in tension, the design for the slab can be thinner for a bonded case than for an unbonded case (*2,17–19*).

Although a fully bonded system would be ideal, it has been shown that a partial bond is usually realized as a result of a number of factors (*20,21*). In such a case, the stresses will lie somewhere in between the two extreme cases, as illustrated in Figure 2.

HISTORICAL PERSPECTIVE

According to *NCHRP Synthesis of Highway Practice 99*, the first recorded use of whitetopping in the United States was in Terre Haute, Indiana (*3*). On this project, constructed in 1918, a 3- to 4-in. jointed reinforced concrete overlay was put in place. Between that time and 1992, approximately 200 whitetopping projects had been documented; of which 158 have been jointed plain concrete pavement, 14 continuously reinforced concrete pavement, 10 FRC, and 7 jointed reinforced concrete pavement (*4*). Since 1992, the ACPA has been tracking the use of UTW in the United States and it has documented more than 300 projects during this 10-year period (*7*). Figure 3 displays the historical use of UTW projects in the United States according to the ACPA statistics. The average project, as noted in this figure, is approximately 2500 m^2 (3,000 yd^2), and ranges from 170 to 58 500 m^2 (200 to 70,000 yd^2). It should also be noted that although not as closely tracked as for UTW, the trend for TWT use is believed to be similar, based on earlier historical usage, along with individual citations found in the literature (*4,22*).

The use of UTW and TWT projects has not been limited to this country. The first examples of modern UTW construction were cited in Europe, specifically in Belgium and

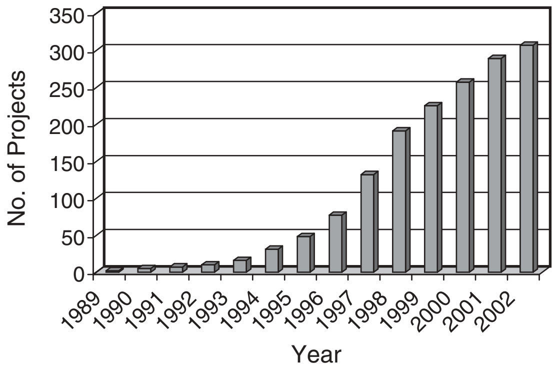

FIGURE 3 UTW construction in the United States.

Sweden (*12,23–25*). Other countries reporting recent projects include Canada (*26*), Mexico (*27*), Brazil (*28,29*), the Republic of (South) Korea (*30*), Japan (*31–33*), France (*34–36*), Austria (*37*), and The Netherlands (*38*).

During the mid-1980s, a concept termed "fast-track" concrete paving became more widely known (*39*). This method involves the use of high early strength concrete, thicker slab design, and a number of other construction techniques, allowing concrete pavements to be opened to traffic in as little as 12 h after placement (*39,40*). This method is of particular benefit on UTW and TWT projects, because it demonstrates that these overlay types can be cost competitive and still constructed with minimal disruption to the traveling public.

EXAMPLES OF USE

The application of UTW and TWT overlays has been widespread. Of those respondents to the synthesis survey, 40% have worked with these overlays within the last year, with two-thirds of the projects in the UTW category. As mentioned previously, this type of rehabilitation has been used for more than 80 years, although it has only more recently been advanced as a viable solution in modern HMA pavement rehabilitation.

From the literature, it is evident that both UTW and TWT overlays have been successfully used to rehabilitate existing HMA pavements nationwide. In Tennessee, it is reported that "the use of ultra-thin whitetopping . . . has moved beyond its experimental status to become a viable alternative to HMA in certain applications" (*41*). The Florida Department of Transportation (DOT) reports, "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 Weight-Station on I-10" (*42*).

The following is a summary of a number of additional projects cited in the literature. More detail on some of these case studies can be found in Appendix B of this synthesis or in the references cited.

Kentucky

The first high-profile UTW project in the United States took place in Louisville, Kentucky, in 1991 (*6,43*). It was a landmark in UTW history, demonstrating that "ultra-thin whitetopping 50 to 90 mm (2 to 3.5 in.) thick can carry traffic loads typical of many low-volume roads, residential streets, and parking lots" (*10*). This project was constructed of FRC panels, which were instrumented to provide data on the effect of heavy wheel loads.

The pavement section served as an access road to a disposal facility, which was selected to provide performance data on the common distresses and failure modes of UTW overlays that are subjected to heavy loads similar to those of garbage trucks. Because the large number of trucks per week (400 to 600 trucks per day, five and one-half days per week) is 20 to 100 times greater than on lower-volume roads, this project was considered an accelerated test. Some of the significant findings from the Louisville UTW experiment included

- The bond between the UTW and the existing HMA pavement significantly reduces the stresses in the concrete section. This allows the section to perform as a composite section.
- Corner cracking was the predominant distress.
- Joint spacing has a significant effect on the rate of corner cracking.

Virginia

The Accelerated Loading Facility (ALF) at the FHWA Turner–Fairbank Highway Research Center was recently used to test UTW over existing HMA pavements (*44,45*). Eight lanes of UTW were constructed with different thicknesses [64 to 89 mm (2.5 to 3.5 in.)], joint spacings, and concrete mix designs (with and without synthetic fibers).

Design, construction, and performance data were collected for the UTW overlays placed at the facility. Specific data included concrete temperatures, slump, unit weight, and air content. Concrete properties, including compressive strength, elastic modulus, and flexural strength were also tested from numerous cylinders and beams cast during placement. Bond strengths between the PCC and HMA were also tested using a procedure developed by the Iowa DOT (*46*), and stiffnesses of the various pavement layers were estimated by falling weight deflectometer (FWD) testing (*2*).

Data related to distress development and modes of failure have been analyzed as part of a recently completed project (*2,47–49*). From this analysis, it was shown that a major contributor to the failure in the UTW was the resilient versus permanent deformation characteristics of the support layers below the UTW overlays. Specifically, it was found that the more viscous the HMA layer, the more quickly and severely the UTW would deteriorate.

The ALF applies approximately 50,000 loads per week, testing 24 h a day. After several months of testing, 0.7 to 3.5 million 80-kN (18-kip) equivalent single-axle loads (ESALs) were applied. The result was that six of the eight lanes had no significant loss in ride quality (*49*). The two lanes that did exhibit significant distress were constructed on the softest HMA sections.

Iowa

An 11-km (7-mi.) whitetopping overlay project on Iowa's Route 21 between Victor and Belle Plaine consisted of four overlay types representing both UTW and TWT (*162*). Thicknesses on the project ranged from 50 to 200 mm (2 to 8 in.), and some of the concrete was fiber reinforced.

This project yielded performance data on whitetopping with a real highway application. As with other whitetopping research projects, design data, construction techniques, instrumentation, and performance data are all available. Given the large number of combinations of thickness, joint spacing, concrete mix, and surface preparation, this experiment continues to be one of the most referenced pertaining to the demonstration of the applicability of this rehabilitation alternative.

Minnesota

In 1997, the Minnesota DOT (Mn/DOT) constructed two UTW overlay projects. The first was on US-169 in Elk River, where a 75-mm (3-in.) UTW was placed (*50*). A second project included both UTW and TWT ranging from 75 to 150 mm (3 to 6 in.), constructed on I-94 at the Mn/ROAD test facility (*51,52*). These sections were heavily instrumented, which allowed measurement of the static and dynamic response of the pavement under various applied loadings and environmental conditions. The goal of these projects was to determine design features that can optimize the life of the overlay. The results of these projects have better enabled interested parties to understand the behavior and performance of UTW overlays under very controlled conditions.

Missouri

Both UTW [89 mm (3.5 in.)] and conventional [250 mm (10 in.)] whitetopping overlays were placed at the Spirit of St. Louis Airport in 1995 (*8,53*). Although the project was not a highway application, valuable information was collected, which has resulted in improved design and construction specifications. The ACPA continues to monitor the whitetopping overlays at the airport to determine bond strength and stresses in the concrete and in the HMA layer below the overlays.

Colorado

Santa Fe Drive, Denver, Colorado

In May 1996, two 152-m (500-ft) test sections were constructed at this location (*17,20*). These sections included a 100-mm (4-in.) TWT on an unprepared 125-mm (5-in.) HMA and a 125-mm (5-in.) TWT on a milled 100-mm (4-in.) HMA. On some sections, a new HMA mat was constructed 2 to 3 days before the overlay.

The slabs on both sections were 1.5 m^2 (5 ft^2). A total of three test slabs were instrumented for strain, deflection, and temperature. Load testing on these slabs was conducted in August 1996 with a 90-kN (20-kip) axle load at several times during the day. Cylinders, beams, and cores were taken for thickness verification, PCC–HMA bond shear strength, strength (compressive and flexural), PCC modulus of elasticity, HMA-resilient modulus, and unit weight.

On the basis of observations made on this project, it was recommended that the whitetopping overlay not be placed on top of a newly laid HMA layer. Subsequent work has supported this conclusion, but emphasizes that the properties of the HMA—existing or new—should be considered in the overlay design (*2*).

State Road 119, Longmont, Colorado

On this project, a number of design and construction variables were used, including varying TWT thickness, joint spacing, use of dowels, and HMA surface preparation (*17,20*). A total of five slabs were instrumented on this project for strain, deflection, and temperature. Each slab had a unique combination of the aforementioned design features. Construction was done in August 1996, with load testing being conducted the following month. An 80-kN (18-kip) axle load was used for the load testing at this location. Cylinders and cores were also collected and measured for strength (compressive), PCC modulus of elasticity, thickness verification, PCC–HMA bond shear strength, HMA-resilient modulus, and unit weight. One of the more important findings from this project was the recommendation of tie bars along the longitudinal construction joint. It was found that without the tie bars, the adjacent lanes would separate and move longitudinally with respect to each other.

Montana

Great Falls, Montana

To repair an intersection subjected to heavy rutting and shoving, the Montana DOT constructed a TWT overlay at the intersection of N.W. Bypass and 3rd Street N.W. in Great Falls (*54,55*). Constructed in the fall of 1999, the project has been considered a success in meeting the needs of the Montana DOT.

Approximately 2 years after construction, one section was found to have developed a localized failure. Slab shattering was observed on an area approximately 3 m (10 ft) by 4.5 m (15 ft). In March 2001, the damaged area was repaired by removing the existing panels along with the underlying HMA and replacing it with full-depth concrete. On removal and inspection of the pavement materials, it was apparent that an underlying storm drain had been a source of moisture that led to a weakened subgrade and stripping within the HMA layer. Before placement of the full-depth patch, the subgrade was partially removed and replaced with a select unbound aggregate. Since the placement of the original patch, some additional cracking (shattering) has been observed on additional panels in the vicinity.

Kalispell, Montana

In September 2000, the Montana DOT participated in the resurfacing of a segment of East Idaho Street in the city of Kalispell (*54*). The existing HMA pavement was reported to be heavily distressed, with rutting, shoving, and transverse cracking. A TWT was identified as the viable alternative, constructed by removing 130 mm (5 in.) of the HMA surface and replacing it with an equal thickness of PCC.

The project was built using concrete paving practices typical of other projects in the state. Milling of the existing HMA was done, followed by vacuuming and sweeping to remove the remaining debris that may contribute to debonding. Construction was staged to accommodate both traffic patterns and the selection of paving equipment. During the course of the paving, wash water (slurry) from the sawcutting operations was found to cover the milled surface. As a result, additional vacuuming and sweeping were performed before PCC placement to ensure a good bond. During placement, some locations were found to have insufficient HMA thickness to contribute to composite action. As a result, the HMA was completely removed at these locations, and the PCC was placed on top of the existing base. Thicker sections were also used on areas where additional load transfer was desired.

The mix included 1.8 kg/m^3 (3 lb/yd^3) of polypropylene fibers, which were introduced into the trucks at the batch plant. A Bid-Well deck paver was used to place most of the concrete. Vibration during placement was found to be critical. A single handheld “stinger” type vibrator was used, but the recommendation was subsequently made that two units would have been more helpful.

After placement was complete, one small section of concrete was found to be of poor quality as a result of improper vibration during construction. The panels were subsequently removed and replaced, dowelling the replacement panels into the existing overlay.

Overall, however, the Montana DOT research reported that this TWT project is functioning as intended. Of 4,200 panels, only 12 have exhibited minor cracking, and no debonding is evident. People living adjacent to this project have also reported how quiet their neighborhood has become thanks to the lack of heavy vehicles “bouncing” through the intersections.

CHAPTER THREE

PROJECT SELECTION

The proper selection of candidate projects for UTW and TWT overlays is of paramount importance to their continued use as a viable rehabilitation alternative. This chapter discusses some of the issues related to proper project selection. Methods to optimize the design and construction of the project will also be discussed.

PROJECT SELECTION CRITERIA

Traditionally, PCC has been perceived as a material for new pavement construction, in particular for heavy-duty pavements. However, with respect to pavement rehabilitation, agencies sometimes view HMA overlays as the first option, regardless of the existing pavement structure.

It is within this environment that UTW and TWT overlays are gaining popularity. A number of agencies are beginning to use the same operational and economic criteria to evaluate both HMA and PCC overlay options. The barriers posed by traditional concepts of concrete as only a heavy-duty, long-term solution are coming down, and highway departments are recognizing that to best use their available resources, they must program a variety of projects meeting a range of service lives—a so-called "mix of fixes" (*56*).

From the results of the survey, it was found that both the initial cost and longevity of the rehabilitation are the most important drivers in the decision-making process, with traffic control a close third. In order of decreasing importance, other factors included LCC, improvement to smoothness, improvement to texture (safety), geometric design, agency experience, contractor experience, and improvement to noise.

For an existing HMA pavement nearing the end of its structural or functional life, an agency's selection of the most appropriate rehabilitation alternative can be based on those criteria and a number of others. In many cases, the selection of the most appropriate strategy will be made by balancing several competing factors. Common factors include the following (*1,13,15,57*):

- Projected traffic loading
- Existing pavement
  - condition
  - layer thicknesses
  - drainage
- Costs
  - overlay construction cost
  - total LCC
  - user delay costs
  - vehicle operating costs
- Time factors
  - number of construction operations
  - total construction time
  - repair and maintenance time
  - frequency of repair and maintenance
  - initial performance period
- Corridor impact
  - noise level
  - excess pollution level
  - accident rate (vis-à-vis skid resistance)
  - ride quality (smoothness)
- Material availability
  - cement
  - asphalt binder
  - aggregates
- Contractors
  - availability (capacity)
  - experience
  - competition (number of bidders).

Quite often, the design and constructability of the various overlay alternatives will contribute to many of the factors. Therefore, a need exists to assess some degree of engineering design in the planning and selection stages.

For example, one potential pitfall in selecting a UTW or TWT alternative is the result of the oversimplified characterization of the properties of the existing HMA pavement. Depending on these properties, the UTW or TWT can have a widely varied performance. It has been shown that all else being equal, the stiffness of the HMA layers can have a significant impact on the performance of these types of overlays (*2,48,49*). Conversely, an HMA overlay is sometimes prone to the redevelopment of certain distresses, such as shoving at intersections. This consideration has been specifically cited in the survey responses, as well as in the literature, as a reason to select a whitetopping alternative (*58*).

Another consideration that is commonly reported in the literature pertains to the benefits of a more reflective surface as a result of the lighter color of concrete. The increased reflectivity has been reported to have a number of benefits, including

- Increased reflection of headlights and aircraft landing lights, improving safety (*59,60*);
- A lower demand for external lighting, reducing operational costs (*61*); and
- A cooling effect owing to lower absorption of solar energy, with environmental benefits (*61*).

Finally, another reported benefit is the resistance to fuel spillage, which is a possible consideration in the construction of parking lots, fueling stations, and aircraft aprons (*53*).

COST ANALYSIS

The results of the user surveys collected during this synthesis have concurred with the literature that cost is the most significant decision-making criterion (*56*).

LCC techniques are used by some agencies (27% of those surveyed) to compare alternative rehabilitation strategies on an equivalent basis (*62*). Because different overlay strategies can have a range of anticipated lives, two indicators of LCC are commonly used when comparing alternatives:

- Net present value (NPV)—a total cost of the cash flow with all costs discounted to present-day equivalents in dollars.
- Uniform Series Capital Recovery—an equivalent annual outlay of the alternative that, when discounted, is equal to the NPV of the alternative. This technique allows for a better comparison of unequal analysis periods.

In many cases, LCC includes the prediction of both agency and user costs associated with the various analysis strategies (*62*). Although agency costs are straightforward to calculate, determining the costs to the users can be more involved. These costs can include delay, excess fuel and oil expenses, and even driver frustration. Potential societal costs such as noise, air quality, local business access, and others could also be considered.

Methods of estimating the impacts include computer programs such as MicroBENCOST, QUEWZ, TOMCAT, and others (*63–65*). These computer programs require knowledge of the lane closure configuration, estimated traffic flow, and other inputs to estimate the user costs in regard to delay, fuel, and oil. QUEWZ can also estimate excess emissions, as well as vehicle maintenance and depreciation costs (*64*).

A discount rate (calculated from inflation and interest rates) is also needed in an LCC analysis to account for the time value of money. Other considerations in the LCC analysis include maintenance costs and salvage value. Additional guidance on the particulars of LCC is available (*62,66*).

Although LCC is most commonly used in planning to select the overlay type, Figure 4 illustrates how this technique can also assist in the strategy refinement process. In this example, the whitetopping thickness is the only variable different from alternative to alternative. As a result of the LCC prediction, a thickness of 150 mm (6 in.) is found to be optimum. A thinner section, although with a lower initial cost, has a higher NPV owing to the higher degree of maintenance associated with it. Conversely, a thicker section has a high initial cost that makes it prohibitive, even though its maintenance schedule is small.

Although this example demonstrated that LCC can be used to optimize whitetopping thickness, it should be noted that it can also optimize more difficult and unconventional variables such as the joint spacing and fiber content, as shown in Figures 5 and 6, respectively. In these examples, the LCCs of a number of discrete strategies are determined. As illustrated, a short joint spacing or high fiber content will have a high initial cost. Conversely, a long joint spacing or low fiber content may rapidly fail, increasing maintenance and decreasing user satisfaction. In both extremes, a high LCC will result.

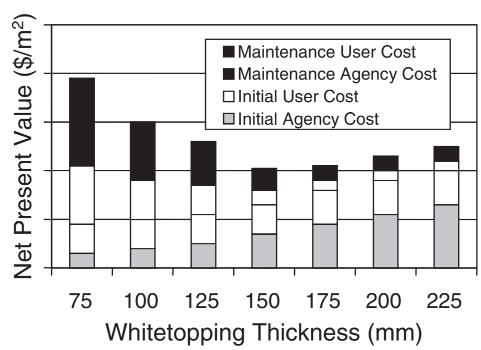

FIGURE 4 Example of LCC comparison of various whitetopping thickness strategies.

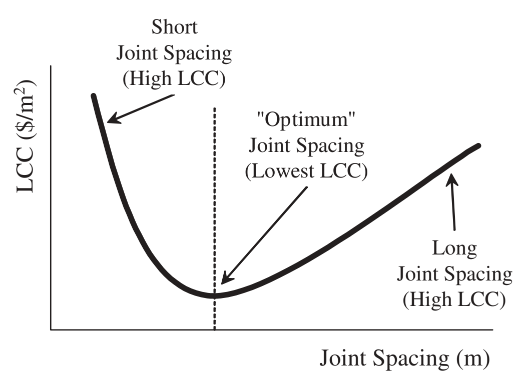

FIGURE 5 Optimizing joint spacing using LCC.

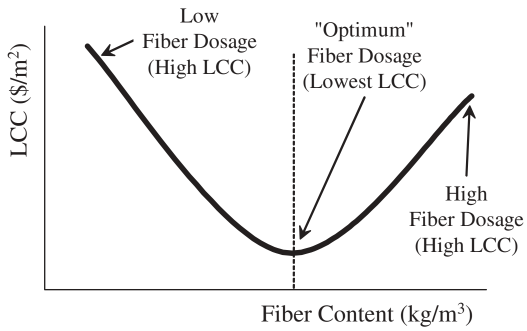

FIGURE 6 Optimizing fiber content using LCC.

The optimum of these factors can be found between the two extremes.

Finally, it is worth noting that whereas consideration of LCCs continues to be encouraged, initial (construction) costs appear to be the dominant decision-making factor. However, by recognizing that UTW and TWT overlays do not necessarily require the same design and construction elements that a 30- to 50-year concrete pavement would require (e.g., stainless steel dowels), one can institute cost-saving measures to improve the overall initial costs of this rehabilitation alternative (*67*).

CHAPTER FOUR

DESIGN

This chapter includes discussion of the various issues related to the design of TWT and UTW overlays. Information is presented in a fashion suitable as either a lookup reference or as a learning tool. Because many aspects of the design of UTW and TWT overlays are similar to those of a conventional concrete pavement design, the discussion provided will not include sufficient detail to serve as a stand-alone design aid. Instead, it can be used alongside other design tools, especially those cited herein, including the *AASHTO Guide for Design of Pavement Structures* (*11*) and the PCA *Design and Control of Concrete Mixtures* (*68*).

EXISTING PAVEMENT

It has been reported that a key to the success of any concrete pavement, including a UTW or TWT overlay, is a uniform and stable support system (*69*). In such a case, the support is provided by an existing HMA pavement. Therefore, it should be recognized that any contributing factor to the failure of the HMA pavement might similarly lead to a failure of the overlay.

A thicker concrete overlay should be considered if the support layers have exhibited poor structural support, by contributing to the deformation and/or cracking of the original HMA pavement (*70*). When designing the UTW or TWT overlay, consideration should be made of the condition of the existing HMA pavement.

In some cases, this consideration includes the use of a reduced stiffness value for the HMA. Rational adjustment factors for a decrease in the HMA stiffness owing to damage have been used in other pavement design approaches, and they could be used in whitetopping overlay design as well (*2,71*). Characterization of the in-place HMA can be found through visual inspection, backcalculation of FWD data, or testing of cores collected from the candidate project. Laboratory testing can include wheel-track testing (e.g., Hamburg) and resilient or dynamic modulus measurements (*1,47*). The synthesis survey revealed that 90% of the respondents relied on some form of visual inspection for characterization of the HMA layer, 62% used pavement management data, 45% used FWD data, and 7% employed laboratory testing.

Recent studies have indicated that the susceptibility of the HMA to permanent deformation may be a significant factor in UTW and TWT performance; therefore, testing for this property may be of benefit, if means are readily available (*2,48,49*). The survey revealed that 76% of the respondents included rut-depth measurements in their design process. In the near future, consideration could be given to employing the simple performance test technology currently being developed and evaluated in *NCHRP Reports 465* and *513* (*72,73*). The devices being considered measure more fundamental properties of the HMA mixture, making extrapolation to a wide variety of loading conditions possible. Although this test is being developed as part of the Superpave system, it shows promise as an improved means to predict the performance of whitetopping overlays (*2*).

In addition to considering the HMA stiffness, another design approach is to adjust the effective thickness of the existing HMA layer. The 1993 AASHTO design guide provides guidance employing this approach (*11*). Adjustment factors can be derived from a visual condition survey, remaining life analysis, or FWD backcalculation.

The stiffness of the pavement system as a whole (including the HMA layer and support layers) is known to have a significant effect on the performance of the whitetopping overlay. As a result, FWD testing should be carried out if possible. Suggestions for the proper interpretation of the data include the procedures documented in *NCHRP Report 372* and elsewhere (*69,74*). With proper characterization of the pavement support system, the reliability of the overlay design will be improved.

PREOVERLAY REPAIR

Before overlay, it is recommended that consideration be made to repair the existing pavement, to provide the necessary uniformity to the support system. Although many existing HMA distresses will not reflect through the whitetopping overlay as they might through an HMA overlay, some types of HMA distress may lead to different types of failures in the overlay. For example, a localized subgrade failure beneath the HMA that leads to alligator cracking may ultimately lead to faulting, roughness, or a shattered slab in the whitetopping, if left uncorrected.

Of those responding to the survey, 38% do not routinely conduct preoverlay repairs. However, full-depth patching to correct localized subgrade failure was reported as used by 28%, pothole filling by 21%, and crack sealing by 10%.

The degree to which preoverlay repairs are conducted will depend on the design of the UTW or TWT, along with the anticipated use of the facility. Usually, if the facility is expected to carry heavy truck traffic, measures should be taken to minimize weak areas in the existing pavement (*1*). However, if the whitetopping is being used for aesthetic or other functional purposes, and it is not anticipated to carry large volumes of trucks, the overlay can be placed without significant repair.

Cost is a common consideration when determining the degree of preoverlay repair (*1*). The additional cost of repair may result in a reduced thickness, because the overall project cost may be fixed. Finally, because concrete will be used for the overlay, preoverlay repairs can be made by simply reconstructing areas with full-depth concrete—replacing instead of overlaying the existing HMA.

The following sections outline some of the more common existing pavement distresses, plus the recommended preoverlay repairs to mitigate them.

Rutting and Shoving

Appropriate action should be taken where moderate-to-severe rutting or shoving exists. The ACPA identifies this situation as rutting greater than 50 mm (2 in.) (*1*). Where only minor rutting or shoving is observed, the distress can usually be ignored. The exception to this situation is when the rutting has occurred within a period of time that was much shorter than normal. In such cases, the HMA (or other support layer) may be highly susceptible to permanent deformation and it should be replaced or otherwise mitigated before overlay.

In some instances, the whitetopping concrete can be placed without correction of the rutting or shoving distress (*75*). Doing so may prove beneficial, because a thicker concrete section will be the result in the wheelpaths, where the traffic-induced stresses are usually the highest.

If corrective action is warranted, milling is the most effective for rutting and shoving problems in the existing HMA. This practice also improves the bond between the PCC and HMA, but care should be taken so that adequate HMA thickness remains after milling, if considered essential in the design (*17*).

If the source of severe rutting is found to be the base or subgrade layers, a whitetopping overlay should be used with caution. If there is no replacement of these layers, the performance of the overlay may continue to be affected by the poor quality of the layers.

Leveling courses have also been used in the past to correct for rutting and shoving, but they are not encouraged for use on UTW and TWT projects unless care is taken in the proper selection of the HMA material. As will be discussed later, the properties of the leveling HMA can affect the PCC–HMA bond as well as the stiffness and stability of the whitetopping support system (*48*). In addition, a leveling course adds a construction operation, which may adversely affect the user delay costs and project budget.

Potholes

The ACPA recommends that small potholes that do not have further widespread subsurface failures be repaired before the placement of any whitetopping layer (*1*). Such distresses can be repaired by filling them with unbound aggregate, cold- or hot-mixed asphalt, or concrete. The surface should be well compacted (consolidated) to reduce further movement by loading that propagates through the whitetopping layer after it has been constructed.

The presence of numerous or severe potholes may be an indication of a weakened pavement structure. Therefore, those who design a whitetopping overlay should proceed with caution because the performance of the overlay will be adversely affected by a weak support system (*70*).

Subgrade Failure

Localized subgrade failures are often evident by surface failures such as potholes and map or alligator cracking (*13*). In these cases, the affected area should be removed and replaced before overlay. All of the material affected by the failure or prone to future failures should be removed and the existing HMA structure reconstructed to its original elevation. Existing compaction levels should be matched to avoid differential consolidation of the area after the whitetopping layer has been placed. It is also possible to design full-depth concrete sections for these areas (*76*). These sections can be placed at the same time that the overlay construction is done. This alternative would eliminate not only the need for the additional construction process of HMA placement but also the possibility of poor support or bond caused by a tender mixture.

Cracking

The ACPA recommends that cracks in the existing HMA pavement be filled if the crack width is larger than the maximum size of the aggregate in the whitetopping concrete mix (*1*). Some researchers have also noted reflective cracking from an underlying HMA pavement through a UTW overlay (*50*). Therefore, if severe cracking is observed, especially thermal cracking, measures may be taken to repair these cracks before overlay. Because cracking can often be extensive, a cost analysis should be considered to optimize the degree of preoverlay repair (*1*).

For thinner whitetopping overlays constructed over thick and stiff HMA layers, it may be advisable to match the joints with longitudinal cracks that may have developed along construction joints in the HMA. If this is done, it is advisable to seal these cracks to minimize infiltration of water through the PCC and HMA layers and into the underlying support system. It may be possible to use flowable (cementitious) fill for this purpose.

PCC–HMA Bond

Numerous sources investigated during this synthesis study concurred that a sound bond between the overlay concrete and underlying HMA is key to the success of UTW (*1,70,77*). The same is true of TWT overlays that have been designed with consideration of a bond (*17*).

The bond at the PCC–HMA interface creates the necessary composite section, lowering the neutral axis so that load-related stresses in the concrete are substantially reduced. Short joint spacings, common on UTW and TWT overlays, further reduce these stresses, because the slabs are not long enough to develop as much bending moment (*78*).

The Iowa Shear Test has been the most commonly reported test for assessing PCC–HMA bond (*46*). Test results for bond strength are typically in the area of 100 psi for a whitetopping considered to have a sound bond (*79*). However, caution should be used when comparing test results measured at different temperatures and loading rates, because the behavior of the HMA will vary significantly owing to these factors (*80*).

Proper bonding will not only reduce wheel-load stresses but it will also resist curling forces responsible for lifting of the panels' edges and corners (*70,81*). The key to a successful bond is proper surface treatment of the HMA layer. Because economic effects are also a consideration, there is commonly a trade-off between assurance of the bond and the associated additional costs. The following sections describe the more common techniques that have been used to develop an interface between the existing HMA and the new PCC overlay.

No-Mill Option

In some cases, a bond has been achieved between the existing HMA and the whitetopping overlay with no mechanical surface preparation, except for cleaning (*79*). Under direct placement operations, the existing HMA layer is not milled. This method can be used when rutting and/or shoving are not severe enough to require milling for correction of the surface. In this situation, the surface of the HMA should be cleaned before placement of the whitetopping layer. Of the survey respondents, 33% have used sweeping for this purpose, 37% used air blasting, and 30% used water blasting.

It should be noted that because this method has only minimal assurances of bond it is discouraged for UTW. However, it can be used on TWT applications, especially if composite action is not a consideration in the design.

In cases where low-severity rutting and shoving are not repaired before the whitetopping overlay placement, the excess quantity of PCC needed to fill the surface distortions should be calculated (*1*). This quantity will be in excess of the amount calculated for the design thickness of the whitetopping overlay. Furthermore, the surface irregularities should be considered when recommending the cross-sectional thickness (*82*). If high points in the existing HMA surface are not considered, the result may be thinner concrete sections that may lead to a localized failure. That situation also presents one reason that payments are commonly based on concrete volume for UTW and TWT overlays (*83*).

Milling

Milling is commonly used and strongly recommended for UTW overlays. Milling is also used successfully for TWT overlays, but it is not mandatory unless PCC–HMA bonding is considered in the design. Of those surveyed, 90% use milling for HMA surface preparation. The benefits of milling include

- Smoothing out surface distortions,
- Finishing the grade line before placing a whitetopping overlay,
- Establishing a cross slope for the new pavement elevation, and
- Establishing a good surface to which the whitetopping overlay can develop a good bond.

Depending on the equipment used, milling operations commonly remove between 25 and 75 mm (1 and 3 in.) from the surface of the existing HMA pavement. Figure 7 shows an

FIGURE 7 Milled HMA surface before overlay.

existing HMA surface that has been milled. The ACPA recommends that care be taken not to remove too much of the HMA surface, because the overlay thickness design sometimes relies on a minimum remaining thickness of the existing HMA layer (*1*).

After the milling operation has been completed, the existing HMA surface should be cleaned to remove any particles that had been milled off. A thorough sweeping is most often used for this purpose.

Leveling Course

Although leveling courses are not recommended to correct the existing HMA surface before UTW or TWT construction, their use has been reported in filling surface distortions up to 50 mm (2 in.) (*1*). Leveling courses may also be required to correct or modify cross slope or to improve surface drainage. If a leveling course is being considered, an alternate design using a thicker PCC section should also be considered, since the cost of the additional PCC thickness may be less than the cost and time of the HMA laydown.

In some reported instances, the new HMA material in the leveling course has been found to further compact and shift under whitetopping surface deflections (*17*). Such deflections, combined with the instability of the new HMA leveling course, can cause premature cracking in the overlay owing to voids created beneath the whitetopping surface. Thus, if a leveling course is being considered in the design, appropriate adjustments should be made to the stiffness of the underlying HMA and/or to the bond efficiency between the PCC and HMA.

Stripping

Because stripping of the asphalt binder from the HMA has been reported as a contributing factor to premature deterioration of some UTW and TWT overlays, the susceptibility of the existing HMA should be assessed before design (*54,84*). This action should be coupled with an assessment of the probability that moisture will be present to cause stripping. If the stripping is expected to be unavoidable, the thickness design of the overlay should be adjusted based on the assumption of little or no bond between the PCC and HMA. Furthermore, consideration should be made to reduce the HMA stiffness, to account for the loss of the binder over time.

TRAFFIC CHARACTERIZATION

Traffic data are an important requirement for the design and analysis of any pavement structure. The number of truck loads or 80-kN (18-kip) ESALs that the pavement must withstand during its intended design life will likely be the most important factor in determining which of the whitetopping classes (thicknesses) should be used (*1*). From the survey responses, 59% of states design whitetopping overlays based on ESALs, 38% from a combination of average daily traffic and percent trucks, and 7% from a more general roadway classification.

Whereas some design procedures employ these more conventional methods of characterizing traffic, recent trends of traffic characterization include those being adopted in the upcoming mechanistic-empirical design guide (NCHRP Project 1-37a) (*85*). The new methods, although not being used by any of the survey respondents at this time, will require additional detail about the anticipated traffic loading, including the following:

- Axle-load spectra—the distribution of the number of axles falling into categories of weight and type of axle. For a given axle type, heavier axles will lead to greater damage in the whitetopping overlay, as they do on other pavement types.
- Seasonal distribution of traffic—allowing for a more accurate assessment of damage as affected by seasonal changes in the pavement structure. For example, during the summer months, the underlying HMA will have a lower stiffness that will have an impact on the structural capacity and bond with the PCC.
- Growth—traffic level will usually increase with time. Adjustment factors are required to reflect the change in traffic loading over time.
- Time of day—an important consideration being the deflected shape of the pavement that will differ from day to night, owing to curling. This behavior can affect the predicted traffic-induced stresses. Furthermore, the temperature of the HMA will have an impact on the stiffness and bond strength.

CONCRETE MATERIALS

TWT and UTW overlays are intended for existing HMA pavement rehabilitation. If these types of overlays are to be constructed under traffic, it may be desirable to use "fast-track" concrete materials (52% of those surveyed use these methods). Fast-track construction requires a higher early strength than is required of conventional mixes. However, care should be taken when using such mixes, because a greater potential for shrinkage, and thus random cracking, can exist (*39,86*).

Other considerations should also be made, including mix proportioning, admixtures, and particularly the use of fibers. Engineering properties such as strength are also important, because they are commonly used inputs into the design procedures. This section will describe the more relevant facets of concrete-making materials as they relate to UTW and TWT design.

Aggregate Properties

Because aggregates constitute the largest portion of PCC in both volume and mass, their properties will dominate the overall behavior of the mix (*87*). Some key aggregate thermal properties that affect the behavior and performance of whitetopping include the coefficient of thermal expansion (CTE), thermal conductivity, and specific heat. From a stress perspective, CTE is the most important property. All else being equal, pavements constructed with aggregates of low CTE will perform better than those constructed with aggregates of higher CTE (*88*). This is because higher slab stresses and wider joint openings can occur when aggregates with higher CTE are used.

Another aggregate property to consider is the maximum size of the aggregate. As a rule of thumb, the nominal maximum aggregate size should be no larger than one-third of the slab thickness (*89*). Optimizing the aggregate gradation and shape should also be considered, because an improvement in strength, durability, and workability can result (*90*). Both the top size and gradation of the aggregate will also affect aggregate interlock at the joint, which is another important consideration, because UTW and TWT overlay joints are typically not dowelled.

Cement and Supplementary Cementitious Materials

As opposed to aggregate properties, cement paste properties change as a function of time, especially during the early-age period (*39,91*). Cement Types I and III are most commonly used for fast-track mixes (*39*). Types II and V are occasionally used in overlay construction, especially in areas with high sulfates. Each cement type reacts differently when exposed to water, for each releases different quantities of heat at different rates. Therefore, the type of cement used can affect overlay behavior and performance.

Supplementary cementitious materials (SCMs) such as fly ash and ground-granulated blast furnace slag have been used successfully on UTW and TWT projects nationwide (*2*). Under hot-weather paving conditions their use is encouraged (*92*). Specifications that currently accommodate their use for other PCC paving applications can similarly be adopted for whitetopping.

The water-to-cementitious materials ratio is an important indicator. It is defined as the mass of water in the mix divided by the combined mass of the cement and any additional SCMs, including some fly ashes, silica fume, and slag. This ratio is critical in predicting the overall strength and durability of the mix, as well as other mechanical properties including shrinkage and creep (*68,91*).

Admixtures and Fibers

At the very least, PCC consists of three basic constituents: aggregate, cement, and water. In addition to these basic components, admixtures may be used to enhance or otherwise modify a particular characteristic of PCC (*68*). The use of admixtures can significantly affect the behavior of PCC during the hydration period, especially in fast-track mixes, which have a greater behavior susceptibility to even small variations in mix design (*86*).

It has been reported that the addition of fibers to concrete enhances the toughness and residual strength of concrete (*93–95*). Of those respondents surveyed, 64% always use fibers in their whitetopping mixtures, 32% use them sometimes, and 4% do not use them at all. Fibers may be of particular benefit to whitetopping overlays, because they reportedly reduce permeability, minimize crack width, reduce surface spalling, and increase wear resistance (*94*).

Fibrillated polypropylene fibers are most commonly used on UTW overlays, and they are sometimes specified on TWT applications (*96*). The most common dosage rate follows the example of the Louisville, Kentucky, project, which used 1.8 kg/m^3 (3 lb/yd^3) (*10,43*). Successful projects have also been constructed employing monofilament fibers (typically polyolefin) and steel fibers (*24,97,98*). From the survey, 78% of respondents use fibrillated synthetic fibers, 37% use synthetic monofilament, and 7% use steel fibers.

Tests performed at Gainesville, Florida, indicated no benefit to the addition of fibers; however, it was reported that more loading should be applied to determine if there is a benefit (*70,99*). Other sources have stated that the addition of fibers improves the strength and durability of concrete, and it reduces the occurrence of plastic shrinkage cracking (*100*).

Currently, the industry recommends that enough fibers be added to provide a residual strength of 0.6 MPa (80 psi) when measured in accordance with ASTM C1399 (*94,101*).

Concrete Properties

The concrete mix for a particular whitetopping project is often selected based on the requirements for opening to traffic. For rapid strength gain under fast-track conditions, a common requirement in urban areas, concrete mix designs sometimes include higher cement contents (*39*). Strength development is a critical factor in determining the opening of the pavement to traffic. The 28-day strength of the mix should also be considered, to satisfy the pavement design, including the long-term effects of fatigue and other traffic-induced distresses (*102,103*). The impact that including fibers has on the residual strength should also be addressed (*104*). The designer should consider specifying the use of maturity, because it has been shown to provide better construction control, ensuring a higher quality placement (*105,106*). Maturity is a nondestructive means to estimate concrete strength by monitoring temperature and elapsed time in the field. Finally, the drying shrinkage potential of the concrete mixture should be a factor during the design, particularly with selection of a proper joint spacing. If high shrinkage cannot be avoided, it could be partially compensated for with shorter joint spacing.

CLIMATIC CONSIDERATIONS

Along with traffic, climatic conditions can affect the behavior and performance of a whitetopping overlay system. During construction, the most detrimental effects of climatic conditions occur at extreme temperature conditions, specifically at air temperatures greater than 32°C (90°F) or less than 4°C (39°F) (*92*). The design and construction of the whitetopping overlay should consider the possible effects of both lower-than-normal and higher-than-normal curing temperature on concrete properties and pavement behavior at early ages. Developing specifications that properly account for the interaction between the selected concrete mix and the environmental conditions encountered during construction is essential. Distress types that occur at an early age include random cracking, transverse cracking, and even delamination of the PCC from the HMA. These effects can be minimized with proper temperature control provisions. Documents such as the hot-weather and cold-weather placement guidelines from the American Concrete Institute (ACI) should be consulted (*93,107*).

Climatic effects over the life of the overlay should also be considered. For example, environment-related distresses such as D-cracking and freeze–thaw damage can occur if proper precautionary measures are not taken (*12,68,108*).

WHITETOPPING THICKNESS

The thickness of the whitetopping overlay is a significant characteristic of the overall pavement system. It will drive not only the performance but also the cost. As a result, the selection of the thickness must balance a number of factors, including the anticipated traffic loading, the strength and stiffness of the existing pavement, the properties of the overlay concrete, and the degree of load transfer (*1,2,17,18,77*).

In the thickness design, variability of the various design inputs should be accounted for, if possible (*1*). Reliability is a means by which this variability (as well as variability in the other design inputs) can be rationally considered (*11*). The level of reliability should be determined based on the importance of the facility. The use of certain reliability levels will ensure that the final design can be evaluated based on the selected confidence levels. Not all design procedures include an explicit means to enter variability or reliability inputs, relying instead on built-in factors of safety (*20,77*). In any case, pavement designers should always be cautious when selecting "conservative" values for pavement design inputs, unless explicitly instructed. If the designer takes an overly conservative approach, the resulting design may prove more costly than necessary.

When the whitetopping thickness is selected, additional checks should be performed, including on the remaining overhead clearance under structures, and matching curb and gutter sections. In addition, because whitetopping overlays are commonly preceded by a milling operation, the reduction in HMA thickness should be a factor.

Using whitetopping inlays (deep milling followed by placement of the overlay) can be considered on very thick HMA sections. This is a commonly used technique, because if design is done properly, there are no drainage and overhead clearance problems, the surface is renewed, and additional structural capacity is provided. A more detailed analytical discussion of available thickness design methods appears later in this chapter.

JOINTS

One of the more notable characteristics of UTW and TWT overlays is their shorter joint spacings. Designs using short joint spacing can significantly reduce curling stresses (*109*). In addition, combined with adequate PCC–HMA bond, the shorter joint spacing reduces the flexure (bending) of the concrete panels, further lowering the stress (*110*).

The most common rule of thumb for UTW and TWT panels is to select a joint spacing that is 12 to 18 times the thickness (*1,111*). This range of size was confirmed from the survey responses. Thinner sections are commonly on the low end of this range (i.e., 12), and thicker TWT sections on the high end (i.e., 18). If a TWT is designed and constructed without a bond to the underlying HMA, recommendations for joint spacing increase to 21 times the thickness. The ratio of the length (longest dimension) to width (shortest dimension) of any given panel is recommended to be no more than 1.5, although some recommend a maximum ratio of 1.25 (*1*). It should be noted that given the large number of joints typically specified for a UTW or TWT, the cost of the additional sawing might be weighed against the cost of a thicker PCC section, which would require less sawing.

In UTW and TWT, load transfer is commonly provided by aggregate interlock, which is enhanced by a small joint opening resulting from the short joint spacing (*112*). Aggregate interlock can be further improved by the selection of a concrete mix having a well-graded aggregate with a larger top size (*113*). Furthermore, the joint opening can be reduced (enhancing aggregate interlock) by selecting a mix with both low shrinkage characteristics and low CTE (*112*).

Although most UTW and TWT overlays do not need them, dowels and/or tie bars have been used successfully on some TWT projects, and they are recommended if heavy traffic is expected (*1*). Colorado strongly recommends the use of tie bars to minimize the movement of the longitudinal construction joint (*17*). Iowa has recommended tying the outside panels in cases where little or no support may be provided at the shoulder (*114*). Recommendations for dowel and tie bar size and spacing can be found in a number of industry publications (*1,11,115*).

The designer should also check the expected location of the wheelpath in relation to the joint layout. It has been reported that corner breaks occurred on UTW sections where the wheelpath was located on the longitudinal joints (*10,48*). If possible, longitudinal joints should be designed away from the wheelpaths, preferably having the wheelpath fall in the center of the panel, directly between two longitudinal joints.

If there is an abrupt change in the underlying HMA section, along a lane–shoulder joint, for example, then consideration should be made in matching it with a joint in the thinner UTW overlays. The same is true of any significant transverse transitions—joints on a large patch, for example. In such a case, a transverse joint in the UTW could be aligned to match the patch, even if it falls out of the regular sequence of joint spacing. Because some users have reported reflective cracking from working joints in the HMA, through the UTW overlay (*50*), this may be useful for mitigation.

Sawing of joints on UTW and TWT overlays can be done with conventional sawcutting equipment, but it is more commonly done with early-entry (green) saws. Because of the thinner sections, higher shrinkage potential, and high restraint caused by the bond between the PCC and HMA layers, the potential for early cracking is greater with UTW and TWT compared with conventional concrete paving (*39*). Using early-entry saws minimizes the potential for uncontrolled cracking. Both the literature and the survey responses reported that saw-cut depths should be 25% to 33% of the thickness. Shallower cuts—as little as 25 mm (1 in.)—have also been reportedly used with success when early-entry sawing is used (*116*).

Because of the shorter joint spacing, sealing of joints is typically not performed on UTW or on many TWT overlays (*9*), with only 16% of those surveyed reporting that they seal the joints. However, the user survey also reported that the joints on thicker TWT overlays, commonly containing dowels and/or tie bars, are sealed.

TRANSITIONS AND OTHER DESIGN FEATURES

During the design process, detailing of the transition from the whitetopping to the adjoining pavement should be provided. Transitions between the whitetopping overlay and the adjacent pavement are usually more susceptible to damage owing to a number of factors. For example, horizontal movement of the concrete (from shrinkage and/or thermal contraction) can lead to a wide joint at the transition (*112*). In addition, settlement or shoving of either or both pavement structures can lead to a vertical "bump" or "dip," which in turn produces additional dynamic loading from vehicle excitation (*117*).

To mitigate those effects, the following design features have been used successfully:

- Pavement edge thickening—tapering from the designed whitetopping thickness to the design thickness plus 75 mm (3 in.). The transition typically occurs over a 2-m (6-ft) extent (*111*). This technique was reported as being used by 60% of the survey respondents.
- Transition slab reinforcement—welded wire mesh or two-way reinforcing bar that can be used to reinforce the first 3 m (10 ft) of PCC from the transition (*2*). This technique is used by 7% of the survey respondents.
- Isolation (expansion) joints, typically constructed 12 to 25 mm (0.5 to 1 in.) wide and filled with a preformed compressible joint filler (*13*). These joints are used for transitions to an existing pavement.

EXISTING DESIGN PROCEDURES AND MODELS

This section briefly describes existing procedures for whitetopping overlay design. At least four different design procedures have been identified in the literature, including those from Colorado (*17*), New Jersey (*18*), the Portland Cement Association (PCA) (*19*), and AASHTO (*1*). The following two sections describe the PCA UTW and the Colorado TWT design procedures. Although more advanced procedures have recently been developed (*2*), the procedures discussed in this report are believed to be the most representative of the state of the practice for UTW and TWT overlay design, respectively. Information on the other procedures can be found in the references cited. It should also be noted that the formulation of the models is such that a spreadsheet application could be developed to assist in the design.

PCA UTW Design Procedure

This design method is technically an analysis procedure that allows for the prediction of the number of loads to failure for a given UTW configuration (*19*). The models used are mechanistic–empirical in nature. During their development, three-dimensional (3-D) finite-element methods (FEM) were employed to consider the unusual geometry of UTW overlays.

To verify the 3-D FEM pavement response model, data were collected from three field sites in Missouri and Colorado. When comparing the predictions with the field observations, it was found that the measured stresses in the UTW slabs were approximately 14% to 34% higher than for the fully bonded 3-D FEM simulation. To account for this difference, the 3-D FEM model was adjusted to simulate the partial bonding condition. The stress multiplier selected for the design procedure is 1.36 (a 36% increase), and it implicitly incorporates reliability, because it is the sum of the mean stress difference plus one standard deviation.

The first step in using the design procedure is to calculate the pavement response under load. To do this, a series of regression equations based on the calibrated FEM model were developed. The final equations are as follows:

$$\log_{10}(\varepsilon_{HMA,18kSAL}) = 5.267 - 0.927 \times \log_{10}(k) + 0.299 \times \log_{10}\left(\frac{L_{adj}}{l_e}\right) - 0.037 \times l_e \tag{1}$$

$$\log_{10}(\varepsilon_{HMA,36kTAL}) = 6.070 - 0.891 \times \log_{10}(k) - 0.786 \times \log_{10}(l_e) - 0.028 \times l_e \tag{2}$$

$$\log_{10}(\sigma_{PCC,18kSAL}) = 5.025 - 0.465 \times \log_{10}(k) + 0.686 \times \log_{10}\left(\frac{L_{adj}}{l_e}\right) - 1.291 \times \log_{10}(l_e) \tag{3}$$

$$\log_{10}(\sigma_{PCC,36kTAL}) = 4.898 - 0.599 \times \log_{10}(k) + 1.395 \times \log_{10}\left(\frac{L_{adj}}{l_e}\right) - 0.963 \times \log_{10}(l_e) - 0.088 \times \left(\frac{L_{adj}}{l_e}\right) \tag{4}$$

$$\Delta\varepsilon_{HMA,\Delta T} = -28.698 + 2.131 \times \alpha_{PCC} \times \Delta T + 17.692 \times \left(\frac{L_{adj}}{l_e}\right) \tag{5}$$

$$\Delta\sigma_{PCC,\Delta T} = 28.037 - 3.496 \times \alpha_{PCC} \times \Delta T + 18.382 \times \left(\frac{L_{adj}}{l_e}\right) \tag{6}$$

where

$\varepsilon_{HMA,18kSAL}$ = HMA bottom strain due to an 18-kip single-axle load (με);
$\varepsilon_{HMA,36kTAL}$ = HMA bottom strain due to a 36-kip tandem-axle load (με);
$\sigma_{PCC,18kSAL}$ = UTW corner (top) stress due to an 18-kip single-axle load (psi);
$\sigma_{PCC,36kTAL}$ = UTW corner (top) stress due to a 36-kip tandem-axle load (psi);
$\Delta\varepsilon_{HMA,\Delta T}$ = additional HMA bottom strain due to temperature gradient (με);
$\Delta\sigma_{PCC,\Delta T}$ = additional UTW corner (top) stress due to temperature gradient (psi);
α_{PCC} = thermal coefficient of expansion of the PCC (ε/°F);
ΔT = temperature gradient in UTW (°F);
L_{adj} = adjusted slab length (in.) defined as

$$L_{adj} = 12 \times \left(8 - \frac{24}{\frac{L}{12} + 2}\right) \tag{7}$$

k = modulus of subgrade reaction (psi/in.); and
l_e = effective radius of relative stiffness for a fully bonded system (in.) defined as:

$$\sqrt[4]{\frac{E_{PCC} \times \left(\frac{t_{PCC}^3}{12} + t_{PCC} \times \left(NA - \frac{t_{PCC}}{2}\right)^2\right)}{k \times (1 - \mu_{PCC}^2)} + \frac{E_{HMA} \times \left(\frac{t_{HMA}^3}{12} + t_{HMA} \times \left(t_{PCC} - NA + \frac{t_{HMA}}{2}\right)^2\right)}{k \times (1 - \mu_{HMA}^2)}} \tag{8}$$

where

NA = neutral axis from the top of the PCC (in.) defined as

$$\frac{\left[E_{PCC} \times \left(\frac{t_{PCC}^2}{2}\right) + E_{HMA} \times t_{HMA} \times \left(t_{PCC} + \frac{t_{HMA}}{2}\right)\right]}{E_{PCC} \times t_{PCC} + E_{HMA} \times t_{HMA}} \tag{9}$$

E_{PCC} = modulus of elasticity of the UTW PCC (psi);
E_{HMA} = modulus of elasticity of the HMA (psi);
t_{PCC} = thickness of the UTW PCC (in.);
t_{HMA} = thickness of the HMA (in.); and
L = actual joint spacing (in.).

Once the pavement responses have been calculated through the use of these equations, the next step of the procedure is to calculate the predicted damage as a function of the expected traffic. For this purpose, two modes of failure have been identified in the procedure: (1) fatigue of the PCC at the corner of the UTW and (2) fatigue at the bottom of the HMA.

For fatigue of the PCC, the PCA fatigue cracking equations are used (*100*). For a given stress-to-strength ratio (SR), the number of loads to failure (N_{PCC}) is calculated as:

For $SR > 0.55$

$$\log_{10}(N_{PCC}) = \frac{0.97187 - SR}{0.0828} \tag{10}$$

For $0.45 \le SR \le 0.55$

$$N_{PCC} = \left(\frac{4.2577}{SR - 0.43248}\right)^{3.268} \tag{11}$$

For $SR < 0.45$

$$N_{PCC} = \infty \tag{12}$$

Fatigue damage of the HMA was selected as the second possible failure criterion. For this, the Asphalt Institute method was employed (*118*). The failure criterion for this method is the number of loads that produce cracking in 20% of the wheelpath area. The equation is a function of the modulus of elasticity of the HMA (E_{HMA}) in psi and the maximum strain at the bottom of the HMA layer (ε_{HMA}), as follows:

$$N_{HMA} = 0.0795 \times \left(\frac{1}{\varepsilon_{HMA}}\right)^{3.29} \times \left(\frac{1}{E_{HMA}}\right)^{0.854} \tag{13}$$

The accumulated damage can then be calculated by employing Miner's Hypothesis, which states that failure will occur when

$$\sum_{i=1}^{Load\ Groups} \left(\frac{n_i}{N_i} \right) \geq 1 \tag{14}$$

This analysis procedure can be conducted by dividing the expected traffic loading into load groups with single and tandem axles of known weights. The number of loads to failure in both the HMA and the PCC can be calculated, and Eq. 14 can be used to determine the fraction of the fatigue life that has been consumed. It should be noted that some of the fatigue life in the HMA may have already been consumed by the trafficking before the UTW overlay; therefore, there may be an initial value of fatigue damage for that failure mode.

Benefits

Before the publication of this synthesis report, very little guidance existed on the design of UTW pavements. Other design procedures, such as those in the AASHTO design guide, do not consider the unusual geometry and bond inherent in a UTW overlay (*1*). Additional benefits of such a design procedure are

- Was developed through the use of a 3-D FEM analysis to more realistically predict the pavement responses,
- Recognizes the importance of the bonding characteristics of the UTW to the HMA layer,
- Was validated through the use of data from a number of field sites, and
- Uses multiple failure criteria, recognizing the complexity of the UTW system.

Limitations

Although the approach used in this procedure provides a good foundation for a design method, a number of limitations have subsequently been identified (*2*). The following items require additional consideration:

- To account for the partial bonding between the PCC and HMA layers, a correction term (multiplier) was derived for the corner stress prediction. However, this stress multiplier is based on a single field test location, making extrapolation prone to error. In addition, the use of a constant correction factor oversimplifies the UTW pavement response under load, especially because a constant value for the HMA resilient modulus was used.
- The current design procedure does not account for reliability in the design. The only exception is an inherent reliability of the stress multiplier.
- In the current design procedure, two modes of failure govern the predicted service life: fatigue cracking at the corner of the UTW and fatigue cracking of the HMA layer beneath the UTW. Although HMA fatigue may be a contributor to UTW distress, HMA alligator cracking in the wheelpath, as the model used currently predicts, does not apply, owing to the differences in geometry and loading. Furthermore, the developers stated that for UTW pavements, "the asphalt layer is covered by concrete slabs, and pavement rutting would not be the governing distress." However, from observations of UTW overlays in service, this mode of failure does appear to be significant, and it should possibly be considered.
- The procedure contains the requirements for a number of inputs that may be difficult to define accurately by the typical user including, for example, a "design temperature differential."

Colorado TWT Design Procedure

In Colorado, TWT sections from 125 to 175 mm (5 to 7 in.) thick, with joint spacings of up to 3.7 m (12 ft), were instrumented to measure critical stresses and strains as a result of traffic loads and temperature differentials. These results were used to develop a design procedure for TWT overlays (*17*).

In developing the procedure, theoretical design equations for the prediction of critical stresses and strains in the whitetopping system were devised first. Correction factors were then calculated to account for the experimental data collected. More specifically, the correction factor adjusts the theoretically predicted fully bonded stresses to the partially bonded stresses seen in the field. It also adjusts for the expected temperature gradient.

In this procedure, the location of critical stress is hypothesized as being centered along a longitudinal free edge joint, such as a PCC curb and gutter. Because the joints loaded by traffic will most likely not be in a free edge condition, the design procedure considers tied longitudinal joints. The relationship between the free edge and tied edge stress is given as

$$\sigma_{FE} = 1.87 \cdot \sigma_{TE} \tag{15}$$

where σ_{FE} is the longitudinal free joint load-induced stress and σ_{TE} is the longitudinal tied joint load-induced stress.

Comparison of the theoretical stresses with the measured tied edge loading stresses showed that the measured stresses are greater than the theoretical stresses. The correction factor was reported as

$$\sigma_{EX} = 1.65 \cdot \sigma_{TH} \tag{16}$$

where σ_{EX} is the measured experimental partially bonded interfacial stresses and σ_{TH} is the theoretical fully bonded interfacial stresses.

To assess the PCC–HMA interface, the strains were measured in the top of the HMA pavement and also in the bottom of the PCC slab in the field. It was found that the strain in the HMA is less than the strain in the PCC pavement. An equation relating these was given as

$$\varepsilon_{AC} = 0.842 \cdot \varepsilon_{PCC} \tag{17}$$

where ε_{AC} is the strain in the HMA surface and ε_{PCC} is the strain in the PCC bottom.

Because the coefficient in this equation is not 1.0 demonstrates that there is only partial bonding between the PCC and the HMA.

The effect of the changing temperature gradient on the load-induced stresses was also assessed. The following equation was given to report the change in the stress as a function of the temperature gradient:

$$\sigma_{\%} = 4.56 \cdot \Delta_T \tag{18}$$

where $\sigma_{\%}$ is the percent change in stress from zero temperature gradient and Δ_T is the temperature gradient, °F/in.

As a result of this analysis, design equations were developed for the prediction of PCC stresses from both 90 kN (20 kip) single-axle loads and 180 kN (40 kip) tandem-axle loads. Similar equations were also developed for the HMA strain. All of these equations depend on the effective radius of relative stiffness for the fully bonded slabs. Adjustments are made to the stress equations to account for partial bond and the loss of support as a result of temperature-induced curling.

Both PCC and HMA fatigue relations were used as failure criteria in this procedure. The number of allowable load repetitions for the PCC is a function of the flexural stress-to-strength ratio (*SR*). For the HMA, the number of allowable loads is a function of the maximum tensile strain in the HMA layer, the HMA modulus of elasticity, and the volume of binder and air voids. The amount of fatigue damage that the HMA has sustained before whitetopping construction is also considered. A subsequent review of this procedure has revealed the following (*2*):

- TWT overlay thickness is substantially dependent on the subgrade modulus of reaction.
- TWT thickness is also sensitive to the HMA stiffness and thickness. It was stated in the procedure that the thickness of the HMA layer should not be less than 125 mm (5 in.).
- TWT thickness is not sensitive to the number of 80 kN (18 kip) ESALs for greater than 1 million applications. However, when the loads increase, the stiffness of the HMA becomes a more significant factor. For traffic loading greater than 4 million ESALs, TWT should not be specified if the HMA modulus is less than 2.8 GPa (400 ksi).
- No guidance is provided on the proper selection of the design temperature gradient.

CHAPTER FIVE

CONSTRUCTION PRACTICES

This chapter explores the various facets of TWT and UTW overlay construction. A synthesis of best practices has been summarized for the various topics listed herein. Overall, the best practices for constructing whitetopping overlays are nearly identical to those for constructing any concrete pavements. However, some of the guidance provided here is more applicable to whitetopping overlays.

PREOVERLAY PREPARATION

Before placement of the whitetopping overlay, existing distresses in the existing HMA pavement should be repaired according to the design specifications. For more information on this, refer to the "Preoverlay Repair" section in chapter four of this synthesis.

After all repair activities have been completed, the existing HMA surface should be cleaned to remove any significant dirt or other debris that may be detrimental to the bond between the existing HMA layer and the new PCC surface. Sweeping is the preferred method. Compressed air or water has also been used, but it is not cost-effective or necessary for whitetopping applications (*1*).

Under extreme conditions, solar effects can significantly heat and dry the existing HMA before placement. To mitigate this situation, measures could be considered to cool the surface before placement. Light water fogging is the recommended practice, and it can be used when the surface temperature of the existing HMA is uncomfortable to an open palm (*1*). Just before PCC placement, the HMA surface should be lightly moistened to prevent water from being drawn from the fresh concrete (*1*). A loss of water in the concrete mix at this critical location could reduce the level of bonding between the two layers. However, care should be taken to ensure that pools of water are not present on the HMA surface before placement. Should that happen, debonding can occur, because a weak layer may be formed within the concrete at the bond.

Although detrimental to the bond between the HMA layer and the new PCC overlay, whitewashing has been recommended when the temperature of the HMA surface is expected to exceed 45°C (110°F) (*1*). Note that this practice is not recommended for UTW and most TWT projects. It should be used only on TWT overlays that are not designed to bond to the underlying HMA pavement.

If used, whitewashing should be made of lime slurry (*83*). Lime slurry is made using water and hydrated lime. Chloride salts should not be added to the mixture, even if the directions suggest doing so. It can be particularly harmful to reinforcing steel (e.g., tie bars or dowels) and to the PCC itself. Curing compound has also been used for a whitewash, but it is even more detrimental to the bond between the HMA and whitetopping layers and is therefore not recommended. In short, the preferred surface condition of the HMA before whitetopping is a clean and milled surface with no standing water.

CONCRETE BATCHING AND TRANSPORTATION

Batching operations for whitetopping concrete are the same as for conventional paving concrete. The mix design selected for the project should be adhered to as closely as possible. Additional recommendations on the concrete mixture can be found in chapter four of this synthesis.

From the survey responses, the only notable difference may be that UTW overlays are commonly constructed with ready-mix concrete. As a result, control over the batching process may not lie with the paving contractor. It is therefore recommended that the batch tickets be collected from the trucks before placement and compared with the approved mix design. Additions of mix water not approved by the engineer should be avoided, because they may adversely affect the strength and durability of the pavement.

If fiber reinforcement is used in the concrete, measures should be taken to minimize the possibility of fiber balling. AASHTO recently published the findings of a task force addressing the use of fibers in paving concrete. The reader is encouraged to follow the guidelines outlined in the AASHTO document (*94*). Among the survey respondents, 68% reported that they add fibers to the wet mixture by means of bags or bundles, 16% add fibers to the wet mixture in loose form, and 8% add fibers to the dry constituents before mixing.

Special care should be used in transporting concrete from the batching facility to the project site. It is during transportation, handling, and placement that segregation in concrete can occur.

If the concrete is placed by truck in front of a slipform paver, the concrete should be placed in small, overlapping piles that minimize lateral movement of the material by the paver's auger. Care should also be taken not to track paste or dirt onto the surface of the HMA in advance of the paver. Doing so can lead to debonding if the overlay is placed after the paste has dried (*119*). For the same reason, any wash water residue from adjacent sawing operations should be cleaned before placement.

PLACEMENT TECHNIQUES

Equipment

For whitetopping construction, several choices are available with respect to equipment and placement techniques. According to the user survey, where whitetopping sometimes differs from conventional concrete placement is the use of more compact and less costly techniques. For example, whitetopping inlays have been successfully constructed with either hand placement or slipform paving. For overlays with exposed edges, hand placement with fixed forms or slipform paving is available. The following sections expand on these methods.

Fixed Forms

For fixed forms and hand placement, the forms must be rigidly secured to support the concrete and construction equipment so that settling or other moving does not occur. The forms must also support the lateral pressure of the concrete as it is placed from a ready-mix truck, pumper, or other means. An example of using fixed forms is shown in Figure 8.

FIGURE 8 Use of fixed forms.

Forms should be cleaned and lightly oiled before each use, and cleaned again after each use. They should be removed carefully after the concrete has gained sufficient strength. Care must be taken to prevent damage to the concrete edges and corners (*120*).

Concrete should be placed on the existing HMA pavement as evenly as possible to avoid segregation and to minimize the lateral force necessary in additional spreading as the work progresses. A mechanical spreader or strike-off screed that rides on the forms can be used (see Figures 9 and 10). Handheld vibrators ("stingers") should also be used to ensure adequate consolidation along the forms and at other discontinuities (e.g., manholes or drains).

If a finishing machine is not used, a vibrating screed or roller screed can be used to strike off and consolidate the concrete. In addition, handheld stinger vibrators should be used, because screeds alone may not provide adequate consolidation at the bottom of the overlay. Mechanical spreaders and strike-off screeds can also be used for inlays, where the equipment can ride on the edges of the existing pavement, which describe the finished elevation.

If floating is used, it should be kept to a minimum (see Figure 11). If the concrete surface cannot be finished without the continuous use of floats, changes should be made to the concrete mix or to the finishing machines (*1*).

Slipform

Most slipform pavers spread, consolidate, screed, and float finish fresh concrete in one operation, without the use of fixed forms. Concrete is placed in front of, or fed from the side of, the paver by ready-mix trucks, pumping machines, or other

FIGURE 9 Vibrating screed.

FIGURE 10 Rolling screed.

FIGURE 11 Floating.

means (Figure 12). Slipform pavers usually require stringlines for horizontal and vertical grade control (Figure 13). Where possible, dual stringlines should be used to enhance grade control and smoothness. Because many slipform pavers employ automated floats, additional floating is commonly specified, although often not required behind the paver.

FIGURE 12 Use of a slipform paver.

FIGURE 13 Stringline for guiding paver.

Equipment Inspection and Placement Uniformity

If batching and mixing equipment is used on site, it is important to maintain and inspect the equipment at regular intervals. Batch size, constituent quantities, mixing time, mixer speed, blade wear, and other aspects of on-site batching and mixing should be watched carefully.

Also, as the concrete is placed on the existing HMA surface, it should be placed evenly across the width of the paving area. Doing so will minimize the extra effort of the paver, vibrating or rolling screed, and finishing labor required. If care is not taken, poor consolidation can result, as shown in Figure 14.

Dowels

When dowel baskets are used on TWT projects (see Figure 15), their placement must be coordinated with the movements of the paving train. If there is adequate space to the side of the paving lane where the dowel baskets are placed, baskets can be positioned well in advance of the paving train. Where there is not adequate space, baskets can be placed and secured after trucks have delivered the concrete to the area.

As the paving operation moves over the dowels and baskets, care must be taken not to move the baskets with the force of the concrete or paving equipment. It can sometimes be difficult to securely anchor the dowel baskets to the underlying HMA. Misalignment in the dowel basket can result in a locked joint or premature cracking in the two slabs adjacent to the joint (*121*).

A potential problem with the use of baskets is voids beneath the dowels and the basket steel. The contractor must ensure that the concrete is consolidated adequately to eliminate voids that may form beneath the dowels held in a basket, but that it not be overvibrated, which would cause segregation or excessive loss of air.

FIGURE 14 Poor consolidation.

An alternative to dowel baskets is the use of a dowel bar inserter (DBI) (Figure 16). In some cases, this alternative can save time and money compared with placing and aligning dowel bar baskets. However, the contractor must take extra care to ensure that the DBI inserts the dowel to meet the specified tolerances and does not leave marks on the overlay surface, even after final floating and finishing. When a DBI is used, the concrete around the dowels must be well consolidated after insertion. Additional finishing may also be required to repair the surface of the concrete after the dowels have been inserted.

FIGURE 15 Concrete being placed over a dowel basket.

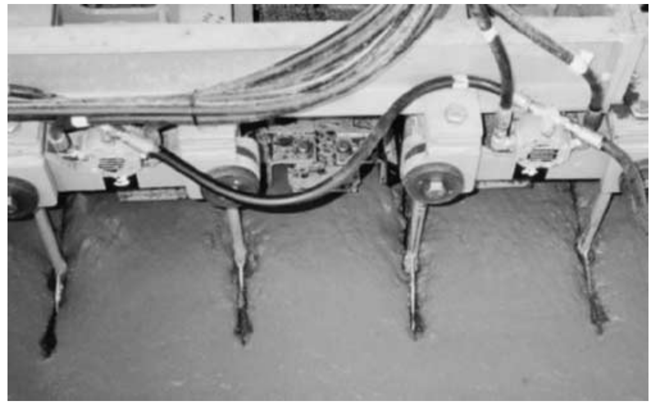

FIGURE 16 DBI submerging dowels.

Other Considerations

Rain Protection

An important practice when paving is to maintain a contingency plan in case of inclement weather. If more than a light rain is expected, construction should be stopped, and fresh concrete already placed should be covered for protection (*122*). Failure to do so can result in the concrete surface losing significant strength and prematurely spalling and/or abrading. If at any time fresh concrete is found to have standing surface water, the finishing operations should be delayed until the water has evaporated or otherwise been removed from the surface. Floating or troweling the excess surface water into the fresh concrete will change the water–cement ratio at the surface, adversely affecting the durability of the surface and leading to spalling or other surface defects.

Thickness

When placing whitetopping concrete on uneven, rutted, or shoved HMA, the concrete should be placed so that the thinnest part is equal to the design thickness (see Figure 17) (*82*). The surface elevation of the new whitetopping surface should be constructed to meet the requirements of the design and plans.

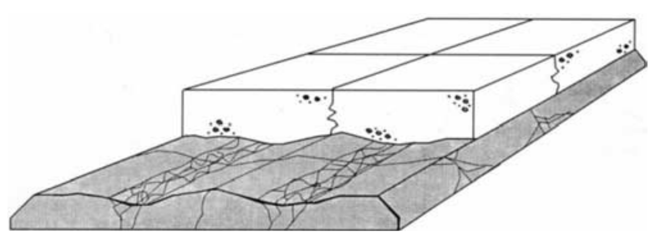

FIGURE 17 With an uneven HMA, err on the thick side (*82*).

Smoothness

Almost all states now require the pavement surface to meet specific smoothness criteria (*123*). An important method for producing smooth whitetopping overlay surfaces is to maintain consistency of the concrete throughout the duration of the project (*124*).

The use of dual stringlines with a slipform paver can also improve the smoothness of the whitetopping surface (*124*). Dual stringlines can act as a redundant system, minimizing impact if one string is inadvertently moved or bumped. Dual stringlines also provide excellent grade control.

The forward motion of the paving operations is also a key to improving smoothness. Any interruptions in the concrete delivery or forward motion of the screed or slipform paver can lead to a bump in the surface of the overlay. Owner-agencies are encouraged to specify end-result requirements for smoothness. Doing so will allow innovative grade controls that can improve smoothness, including those employing lasers, ultrasound, or global positioning system technologies.

TEXTURING AND CURING

Surface Texturing

Timing

Concrete overlay surface texturing operations should be performed at just the right time, to not disturb the curing or setting of the concrete. Texturing should be performed just after the water sheen has disappeared, but before the concrete becomes nonplastic. The water sheen can be seen in Figure 18.

Texturing Methods

Depending on the facility type and traffic speeds, several different methods of concrete texturing can be used. For example, during common wet-weather conditions, and for highway speeds, vehicles require the additional tire–pavement interaction provided by increased texture in the overlay surface. For lower-speed facilities, alternate texturing methods may be used. The following sections describe texturing methods that have been used on UTW and TWT projects.

FIGURE 18 Water sheen on a concrete surface.

Burlap or Turf Drag For lower-speed facilities such as city streets, residential areas, or other urban or municipal applications, a lesser degree of texturization is required (*125*). One potential exception would be whitetopping at intersections where transverse texture may be desirable to improve stopping skid resistance. Texture applications such as burlap drag (Figure 19), turf drag, or a coarse broom (Figure 20) may be sufficient for these projects.

With a burlap or turf drag, the material is often dragged from the paving train across the entire paving width. With a coarse broom, as shown in Figure 20, the broom is often pulled transversely one broom width at a time, along the entire length of the freshly placed overlay. Uniformity should be emphasized when using this type of texture.

Tining For high-speed facilities, such as highways, freeways, and higher-speed urban streets, increased texturization is often recommended. This type of texture is created by the use of a tining comb (Figure 21).

It is important, when using a tining comb, to make shallow striations [typically 3 mm (1/8 in.] in the transverse or longitudinal direction. The tining comb should not overlap its previous passes.

Some states have or are considering discontinuing the use of tining owing to possible adverse effects on curing, noise,

FIGURE 19 Application of texture with a burlap drag.

FIGURE 20 Application of texture with a coarse broom.

FIGURE 21 Tining comb texturization on freshly placed concrete.

and long-term durability, instead relying on other techniques such as a drag finish (*126,127*). Detrimental effects observed by some DOTs as a result of tining include a weakened plane in the concrete surface, which may cause delamination, spalling, and popouts. Tining may also cause undue delay in the curing operation. With the advancement of vehicle tire technology in the past 40 years, the interaction between tires and the overlay surface in wet-weather conditions has dramatically improved (*128*).

Curing

The major objective of concrete curing applications is to prevent the rapid loss of water from the concrete. Proper curing of UTW and TWT is particularly important, because these overlays are thin with large surface areas compared with the volume of concrete. As concrete loses water as a result of evaporation from the top surface, differential drying shrinkage can occur (*39*). This is a major contributor to shrinkage cracking, especially in whitetopping overlays where the sections are thinner than in conventional concrete paving. The application of curing reduces the loss of water from the surface of the concrete. It also permits more complete hydration of cement in the concrete itself (*39*). Minimizing evaporation also helps to control the temperature of the concrete during its early-age stage.

Curing operations should begin immediately after the water sheen disappears from the surface and immediately after any texturing operations have been completed. In the case of a curing compound, the membrane formed by the compound should not be disturbed after it is placed.

The HIPERPAV computer program (High Performance Concrete Paving Software), developed by the FHWA, is a useful tool for predicting the effect of various curing techniques (*39,86*). It can model the design, materials, construction, and environmental conditions affecting the concrete during the early age.

Curing Methods

Various concrete curing methods are available and each provides different levels of protection. A single coat of liquid curing compound provides the least protection, but additional coats can improve its performance. According to responses to the synthesis survey, numerous agencies specified a "double application" of liquid curing compound. Polyethylene sheets, cotton mats, and wet burlap can provide additional protection.

Liquid Curing Compound White-pigmented, liquid membrane curing compound is used most often because of its low cost and ease of application. It requires neither substantial labor nor expensive and bulky material, such as cotton mats. Its disadvantages are that it provides a minimal amount of protection and the membrane can be ruptured inadvertently.

The liquid curing compound should be white to avoid excess heat absorption from the sun (see Figure 22) (*39*). In addition, the white color enables construction workers to check more easily for coverage uniformity. The liquid compound must be constantly agitated during application to ensure that the mixture is applied correctly. The curing compound spraying operation should be shielded from the wind throughout the process.

The compound must cover all exposed surfaces, including the sides of the overlay. If joints are formed before curing, care should be taken not to apply the compound into them, for doing so can prevent joint sealant from adhering. For UTW, curing compound should be applied at twice the normal application rate, because of its increased sensitivity to drying shrinkage (*101*). This practice is confirmed by the responses to the user survey.

Plastic or Waterproof Paper Plastic (typically polyethylene) sheeting provides good protection to the concrete from water evaporation from the surface (see Figure 23) (*39*). It requires more labor than liquid curing compound, yet it is not as bulky as cotton mats or burlap. Waterproof paper may also be used in the same manner described here for plastic sheeting, but it is not as common.

FIGURE 22 White-pigmented, liquid membrane curing compound.

FIGURE 23 Polyethylene sheeting used as a curing method.

The plastic sheeting must not have any rips or tears through which water can escape. The sheets should overlap to provide full coverage for the concrete surface. Just as with curing compound, the sheeting should cover all exposed concrete surfaces, including the edges of the overlay. To prevent removal by the wind, weight must often be added to the edges to hold the sheeting in place.

Cotton Mats or Burlap Cotton mats represent a great increase in evaporation protection, both by providing additional moisture, if needed, and by protecting the concrete from ambient conditions such as low humidity, high wind speeds, and high temperatures. Cotton mats and wet burlap must be kept continually moist. When the mats get dry, they can become more harmful than proceeding without them as a result of the "wicking" action that draws moisture from the concrete into the mat.

Weather Considerations

In rapid drying conditions, a light water fog may be necessary to maintain moist surface conditions before the application of curing methods (*92*). Light water fogging can be accomplished during a brief period of time when the concrete surface begins to dry but before the curing operations can begin, such as before the completion of texturing operations.

Ambient weather conditions, such as wind speed, relative humidity, and air temperature, can interact with the temperature of the concrete to cause excessive water evaporation from the concrete surface (*68*). Because different curing methods provide different levels of protection, knowing the amount of protection required is important in determining the method to use. To determine the level of protection required, the ambient conditions and concrete temperature must be known.

A portable weather station that records the ambient conditions and automatically predicts evaporative water from the concrete surface can be an invaluable tool for controlling water loss from the concrete surface (*129*). Such a tool can also provide advanced warning when conditions approach predefined limits of evaporation. In the absence of such a tool, the plastic shrinkage prediction nomograph, found in the ACI guidelines for hot-weather concreting, can be used to identify adverse ambient conditions (*92*).

Other Curing Considerations

If even further protection is needed from evaporation, cotton mats can be covered with plastic sheeting to slow the evaporation of water from the mats, thereby maximizing the protection of the concrete surface. Where paving operations can be completed at night, that measure should also be considered (*39*). The lack of sunlight and the associated heat, combined with a higher ambient humidity, can greatly improve the conditions for lower potential evaporation.

The temperature of the concrete itself has a large impact on the amount of water lost to evaporation. Cooling aggregates before batching with a light mist of water can help keep the concrete temperature slightly lower (*92*).

An evaporative retardant, such as a monomolecular compound, similar to a liquid curing compound, can resist the tendency of water to evaporate when conditions approach an evaporative condition.

JOINT CONSTRUCTION

Sawcut Timing and Tools

Some practitioners involved in whitetopping have reported distresses related to the timing of the sawcutting operation (*10,12,48,109*). The timing of this construction procedure is critical in preventing early-age distress. Sawing joints too early can cause the weak concrete to ravel excessively at the joint. Conversely, sawing too late may allow stresses to build and lead to uncontrolled, random cracking in the slabs (*130*).

The amount of time required after placement before the concrete is ready for sawcutting without joint raveling depends on the mix design, the climatic conditions, and the type of coarse aggregate (*131*). Raveling can be caused when the saw blade catches a particle of coarse aggregate and the surrounding cement matrix is not strong enough to hold it in place while the blade cuts it. Concrete with soft coarse aggregate can usually be cut earlier (at a lower strength) than can the same mix with hard coarse aggregate (*131*).

Concrete joints are usually sawcut between 4 and 12 h after placement. However, in hot weather conditions or with a fast-track mix, this period may be shorter. In cold weather, the opposite is true, with perhaps up to 24 h required.

The appropriate time can be determined by several methods. A common method is the "scratch test," in which a nail is run along the concrete surface to determine if the concrete has hardened (*132*). Comparing the concrete's maturity with laboratory data is another method that can help determine the appropriate time for sawcutting (*106*). A third method, the pulse velocity meter, also requires correlation to laboratory data (*133*). Because the maturity method and the pulse velocity meter are also nondestructive, they can be used several times at the same location to determine the ability of the concrete to withstand a joint sawcutting blade.

The FHWA HIPERPAV computer program may also be useful in helping to predict the appropriate time for joint sawing based on the concrete mix design, construction times, and environmental conditions (*39,86*). As a planning tool, it can help in predicting the window in which sawcutting is effective in preventing random cracking, especially if similar placements have been done with known sawing windows.

The timing of joint sawcutting is important, because at early ages large climatic temperature fluctuations and the slab restraint at the PCC–HMA interface can have critical effects on the slab (*81*). Large temperature fluctuations can cause high curling stresses, as a result of temperature differentials through the thickness of the whitetopping slab. Such temperature variations can also affect the axial displacement and stress in the slab. When the concrete temperature decreases dramatically, the high restraint at the bonded PCC–HMA interface can cause large axial stresses. The result is a high probability for uncontrolled cracking (*86*). Another impact of the existing HMA layer is that it is stiffer than normal base materials; therefore, it can help induce higher curling and warping stresses by decreasing the support beneath the slab as it deforms vertically (*109*).

Longitudinal joints should be sawed as soon as possible after the transverse joints. TWT and UWT overlays are especially vulnerable with the associated short joint spacing, and they should be sawed at least down the slab centerline to relieve the stresses that may build in the transverse direction.

Sawcut Method and Depth

Early entry or "green" sawcutting is a technique that can help to ensure that the joints are cut in a timelier manner (see Figure 24). According to the user survey, this method is more commonly used than is conventional sawing. Early entry saws are readily available and have been developed specifically for early-age sawcutting, minimizing the raveling of the concrete at the sawcut. To ensure that the joint sawcutting operations can be completed, backup saws are recommended in the event of equipment failure.

The depth of sawcut for standard joint saws should be between one-fourth and one-third the thickness of the slab (*96*). Where possible, the depth should be at least one-third of the thickness. If early-age saws are used, the sawcut depth may be less, but it still should not be less than 1 in. (see Figure 25) (*116*). It is also important to maintain some intact concrete for adequate aggregate interlock between adjoining slabs, which is obtained by an adequate crack face rather than by sawcut.

The sawcut depth must be adjusted to account for thickened sections to maintain the appropriate depth (*1*). If the slab thickness varies more than 25 mm (1 in.) from the mean thickness, the sawcut should be made deeper into the slab. Such thickened areas may occur at pavement edges, the crown of cross slopes, transitions between typical sections, or significant distortions in the existing surface. Of particular consideration with whitetopping is where concrete is used to fill ruts in the existing HMA layer.

An alternative to sawcutting longitudinal joints that has been successfully demonstrated on numerous projects in Iowa employs a vibrating knife that is passed through the concrete mix immediately behind the profile pan (*134*). This technique creates a separation in the coarse aggregate skeleton that is replaced with mortar. Natural cracking then occurs along the resulting weakened plane.

Joint Sealing

The purpose of joint sealants is to keep water and incompressibles (e.g., sand) out of the joint and the crack face. On some TWT overlay construction, joint sealant is used, as it is in standard PCC pavement construction (*1*). UTW joints are usually not sealed (*111*). That is, UTW joints are usually placed at a short spacing and do not open as much as joints

FIGURE 24 Sawcutting on a fast-tracked UTW.

FIGURE 25 Sawcut depth using an early-age saw.

associated with longer slabs. For this reason, these joints are typically not sealed, a situation shown in Figure 26.

QUALITY CONTROL AND QUALITY ASSURANCE

Quality Control and Quality Assurance Program

For many projects, a formal quality control/quality assurance (QC/QA) program can be beneficial to the quality of the final product. By measuring and tracking the various measures of quality, changes and anomalies can be quickly identified and corrected before the quality falls below a critical level. Although a QC/QA program is typically employed on projects placed over weeks or months, some of the elements described herein can be used for smaller placements as well, such as for a short weekend project.

In most cases, the QC/QA elements used on UTW and TWT projects are identical to those of other PCC pavement projects. Typical components of a successful QC/QA program include the following (*135*):

- The contractor performs QC and acceptance testing.
- The owner (state highway agency, city, etc.) performs assurance and verification testing.
- Statistical comparisons are used to determine data conformity between contractor and owner test results.
- Incentives and disincentives are used to encourage the contractor to achieve higher quality.
- Both the contractor's and the owner's technicians are required to be trained in the same manner, necessitating that both meet the same standards.

The contractor should also be required to implement control charts to track the processes involved in its concrete production and placement operations. Such control charts should be part of an overall QC plan developed by the contractor (with guidance from the owner) and subsequently approved by the owner. A QC plan describes all the activities that will be performed by the contractor to ensure quality in the final product, the whitetopping overlay (*136,137*).

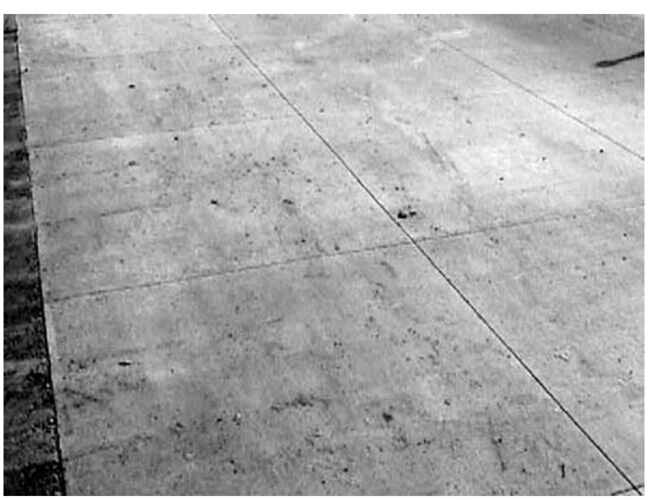

FIGURE 26 Joints without joint sealant.

Recommended QC Tests

In controlling the processes of whitetopping concrete production and placement, the contractor must conduct tests to quantify the level of quality being achieved. Recommended QC (or process control) tests include the following:

- Mixture temperature as it arrives at the site;
- Concrete slump;
- Final water–cement ratio as it is placed;
- Cement factor;
- Admixture dosage;
- Aggregate moisture content, both coarse and fine aggregates;
- Aggregate gradation;
- Fine aggregate fineness modulus or sand equivalent; and
- Unit weight or air content of fresh concrete.

These QC tests should be conducted by the contractor and plotted on control charts whether or not they are required by the owner, simply because doing so provides a clear picture of the state of the process. When there is a graphical representation of the process, adverse trends can be detected early and corrected before becoming critical.

Recommended Acceptance Tests

In contrast to QC tests, acceptance tests measure the finished product; either at the time of placement or after the concrete has hardened (*137*). Recommended acceptance tests include the following:

- Strength
  - Compressive
  - Flexural
  - Splitting tensile
- Thickness
- Unit weight or air content
- Smoothness.

Note that not all of these tests are required for every whitetopping project. In addition, newer tests such as bond strength between the PCC and HMA could be used for QC or acceptance, but they should not be adopted until the specifying agency is comfortable with the techniques and test variability involved.

Payment

By using QC/QA-type specifications, the owner is able to pay for the final product based on the level of quality achieved by the contractor (*135*). For example, when using a percent within limits (PWL) payment methodology, the owner can apply rational pay factors to the unit bid price for the concrete. PWLs and pay factors are described by AASHTO (*83*). Alternative methods for calculating payment, and for PWLs in general, have also been developed by the ACPA and FAA (*101,138*).

Temperature Management

Cold Weather Concreting

There are several methods for protecting concrete in cold weather. The major consideration is to keep the concrete warm enough for the chemical reactions to take place (*106*). The temperature of the concrete is critical during the early-age period. A method for managing the concrete temperature is to compensate for the cold weather-induced heat loss by producing and delivering the concrete at a higher temperature.

General guidelines for ambient temperature include maintaining a minimum temperature of 10°C (50°F) during the first 3 to 7 days and 13°C (55°F) for thinner slabs (i.e., UTW). It is important that the temperature of any surfaces that come in contact with the concrete be above the freezing point of water (*106*).

Maintaining the temperature of the concrete above a minimum level begins with the concrete constituents. The following methods and actions can be beneficial to concrete in cold weather situations (*106*):

- Reduce or eliminate SCMs in the mix. Although SCMs are beneficial under many circumstances, their use in cold weather may delay set and lead to other detrimental effects.
- Increase the cement content in the mix.
- Heat the mixture water and the aggregates before mixing.
- Change the cement type to Type III, if not already specified.

Other methods include using curing blankets or insulated forms. Under extreme cold weather conditions, a heated covering for the whitetopping surface could be provided. Providing an external heat source to concrete slab coverings can slow the escape of heat from the concrete into the surroundings.

Hot Weather Concreting

There are several methods available to protect the overlay in hot weather. The major consideration for the concrete is to maintain the temperature in an appropriate range (*92*). A method for managing the concrete temperature is to compensate for the hot weather-induced heat gain by producing and delivering the concrete at a lower temperature.

Guidelines for hot weather concreting include limiting whitetopping placement to times when the ambient temperature is less than 32°C (90°F). The temperature of the concrete surface should also be limited to 32°C (90°F) during the curing period. This can be accomplished by any of several methods, including shading the concrete surface or paving at night. These are only general recommendations. However, a number of specific actions can be taken to ensure proper concrete temperature control when ambient temperatures exceed 32°C (90°F) (*92*).

As with cold weather concreting, the process of managing the concrete temperature throughout the early-age period begins with the constituents. The following guidelines can help manage the concrete's temperature during the early-age period (*39,92*):

- Type III cement may not be appropriate for hot weather concreting owing to its high heat of hydration.
- It is advisable to use SCMs such as fly ash or ground-granulated blast furnace slag. In addition to controlling early-age temperatures, SCMs usually improve long-term strength and durability.
- Aggregates should be kept cool by storing them in a shaded area or by misting them with water.
- Cool or cold mixing water should be used. In extreme cases, ice has been used—and it is helpful as long as it has been completely melted by the end of the mixing process.
- Water-reducing admixtures can be used to slow the hydration of the cement, thus decreasing the heat generated by the curing concrete.

Other methods include those that protect or affect the concrete either during placement or once it has been placed and are noted here.

- Nighttime paving can help coordinate the maximum heat generation with the coolest time of the day, thus minimizing the potential for internal and external heat to combine and cause a critical situation.
- Where fast-track whitetopping placement is used, paving should be avoided between 10 a.m. and 4 p.m., for the same reasons cited previously.
- The amount of wind blowing across the surface of the whitetopping overlay should be minimized. This can be accomplished by wind screens. Although wind can aid in evaporation, resulting in the cooling of the concrete, it can be even more detrimental when causing the evaporation of needed water, creating a compounded effect of high temperatures and low moisture in the concrete near the surface. The result can be drying, shrinkage, cracking, and other distresses over both the short and long term.
- Application of additional curing measures can keep moisture in the concrete while the hot weather is causing it to evaporate. A double or triple application of curing compound, or even more extensive measures, as described in

the curing section, can help in protecting the concrete during periods of hot weather.

Fast-Track Construction

Many whitetopping projects are constructed under accelerated (fast-track) schedules (*39,139*). This means that the facility is closed to public traffic for only a short period and that the concrete is not very old when it is opened to traffic. Because of these constraints, fast-track mixes are specially prepared to withstand traffic within 1 to 2 days. In addition to high-early-strength mixes, the construction schedule is usually accelerated to maximize the use of the equipment.

Fast-track mixes usually contain increased amounts of Type I or Type II cements or normal amounts of Type III cement to increase the early-age strength gain (*139*). Because an appropriate heat of hydration of the cement is crucial to the concrete strength development, some projects use polystyrene foam insulating blankets to retain heat, where needed. Other methods for increasing the heat development at the appropriate time are used. Fast-track mixes must be carefully designed to generate the right amount of heat at the right time.

Maturity methods are one of the best techniques for controlling strength and temperatures during fast-track construction. Because there are a number of inexpensive systems commercially available, such techniques should be considered on all whitetopping applications (*106*). When maturity is the basis, the FHWA HIPERPAV computer program can also be used to predict the early-age behavior of concrete during the first 72 h after placement (*86*). It can predict the effects of early opening to traffic, as well as help identify the appropriate time for the concrete to be opened to construction or public traffic.

OTHER CONSIDERATIONS

Overhead Structures

When overhead clearance is a problem, the pavement structure may need to be tapered to a full-depth reconstruction and then transitioned again to the typical whitetopping section (*1*). Another option is to raise any overhead structures such as bridges or utilities; however, this is not conducive to the cost-saving and fast-tracking that many whitetopping projects attempt to achieve (*1*). Such issues should be addressed during the project planning stages.

Inline Bridges

The approaches to inline bridges must maintain the same elevation as for the bridge, because the bridge will not be overlaid with whitetopping and because it most likely will not be raised. When the overlay is adding thickness to the pavement, the milling operation should be increased to include a taper of approximately 40 ft per inch of increased depth (*1*). This addition to the milling operation will help to provide a smooth transition when one is approaching and leaving the bridge. Expansion joints at the bridge approaches must be included in the whitetopping overlay.

Pavement Shoulders

The continuity between the whitetopping overlay and pavement shoulders, or curbs and gutters if in an urban setting, must be maintained (*140*). Shoulders must be increased in thickness as necessary (see Figure 27). Curbs and gutters may also need to be replaced or elevated to meet the new elevation of the pavement edge.

Opening to Traffic

Opening to traffic, either construction or the public, must be coordinated with the strength of the concrete and the sawcutting operation, especially in conditions where the facility must be opened to traffic as soon as possible (*139*). Considerations for UTW and TWT in opening to traffic are usually identical to those for other PCC pavement projects. When either the early-strength testing or the maturity method is used, the strength at which the concrete can sustain vehicular traffic can be determined (*106*). Once this strength has been attained, the entire sawcutting operation is complete and the facility may be opened to traffic. Another requirement before opening to traffic includes the resolution of all elevation issues at shoulders or curbs—by permanent solution or by temporary signage and warnings.

When a whitetopping overlay lane must be opened to traffic temporarily, such as for the night or for the weekend, a longitudinal taper must be constructed to allow traffic to drive off the whitetopping overlay onto the existing HMA pavement. When construction resumes, the temporary HMA transition is removed. In addition, it is important that the concrete has gained adequate strength and that the joints have been sawcut before opening to traffic.

FIGURE 27 New shoulders meeting a new whitetopping elevation.

CHAPTER SIX

PERFORMANCE, REPAIR, AND REHABILITATION

EXPECTED PERFORMANCE AND MODES OF DETERIORATION

Since whitetopping was first used, certain predominant distresses have been observed in each class, including TWT and UTW. Research projects, such as the investigation of UTW overlays at the ALF pavements at the Turner–Fairbank Highway Research Center, have provided invaluable information and data toward determining the predominant distresses in UTW overlays (*141,142*). Similar observations have been made of other controlled UTW and TWT overlays nationwide (*18,20,41,42,51*).

Understanding the failure mode of any pavement or overlay type is critical to proper design. One method to accomplish this is using "fault trees." These trees include pathways indicating the steps and processes that eventually lead to observed distresses. Figure 28 is an example of a fault tree for UTW overlays. In this example, several of the common distresses that have been observed are listed at the top of the figure, and the process leading to them is followed by a chain of events.

The reader is encouraged to review the survey responses under "Performance, Repair, and Rehabilitation" in Appendix A. The observations of the various respondents are summarized, and can be reviewed to provide a better understanding of the actual performance of UTW and TWT overlays in the field.

In the following sections, the various types of distress documented in the literature for both UTW and TWT overlays are discussed. As much as possible, the distress types are categorized into early-age and long-term distress types. Early-age distresses are often related to the concrete mix properties, the environmental conditions that occur during construction (hourly effects), and the various techniques employed during construction. Long-term distresses are attributed to the structural design, traffic loading, concrete quality, and environmental conditions that occur after construction (seasonal effects).

Distress Types in UTW

- Early age
  - Uncontrolled cracking as a result of late sawing (*48*), restrained shrinkage, and/or thermal movement (*12*);
  - Plastic shrinkage cracking (*10*); and
  - Joint raveling and spalling as a result of early sawing or opening to traffic too early (*12*).
- Long term
  - Corner cracking (*12,48*), commonly found on sections with large joint spacing and/or significant traffic (*12*). Although this distress is dominant where slab corners are in the wheelpath (*70*), many sections have shown reflective corner cracks on adjacent panels that are not loaded (*48,77*);
  - Debonding and delamination of the PCC from the underlying HMA owing to environmental and/or traffic loading (*10,48*). This has been reported by some to be aggravated by stripping of the asphalt binder from the underlying HMA layer (*2,54*);
  - Longitudinal cracking, especially in the wheelpath (*48*);
  - Transverse cracking on slabs with longer joint spacings (*70*);
  - Fractured and shattered slabs (*48*);
  - Surface wear (*12*);
  - Failure in support layers (*48*);
  - Faulting (*48*); and
  - Spalling (*48*).

Distress Types in TWT

- Early age
  - Uncontrolled cracking;
  - Plastic shrinkage cracking; and
  - Joint edge distresses including spalling and raveling.
- Long term
  - Longitudinal cracking (*143*);
  - Transverse cracking (*143*);
  - Corner cracking (*143*); and
  - Faulting (*4*).

REPAIR AND REHABILITATION SELECTION PROCESS

There are several steps involved in determining appropriate rehabilitation alternatives for a specific UTW or TWT overlay, including the following (*57*):

- If it is not already known, determine the overlay classification—UTW or TWT.
- Evaluate distresses.

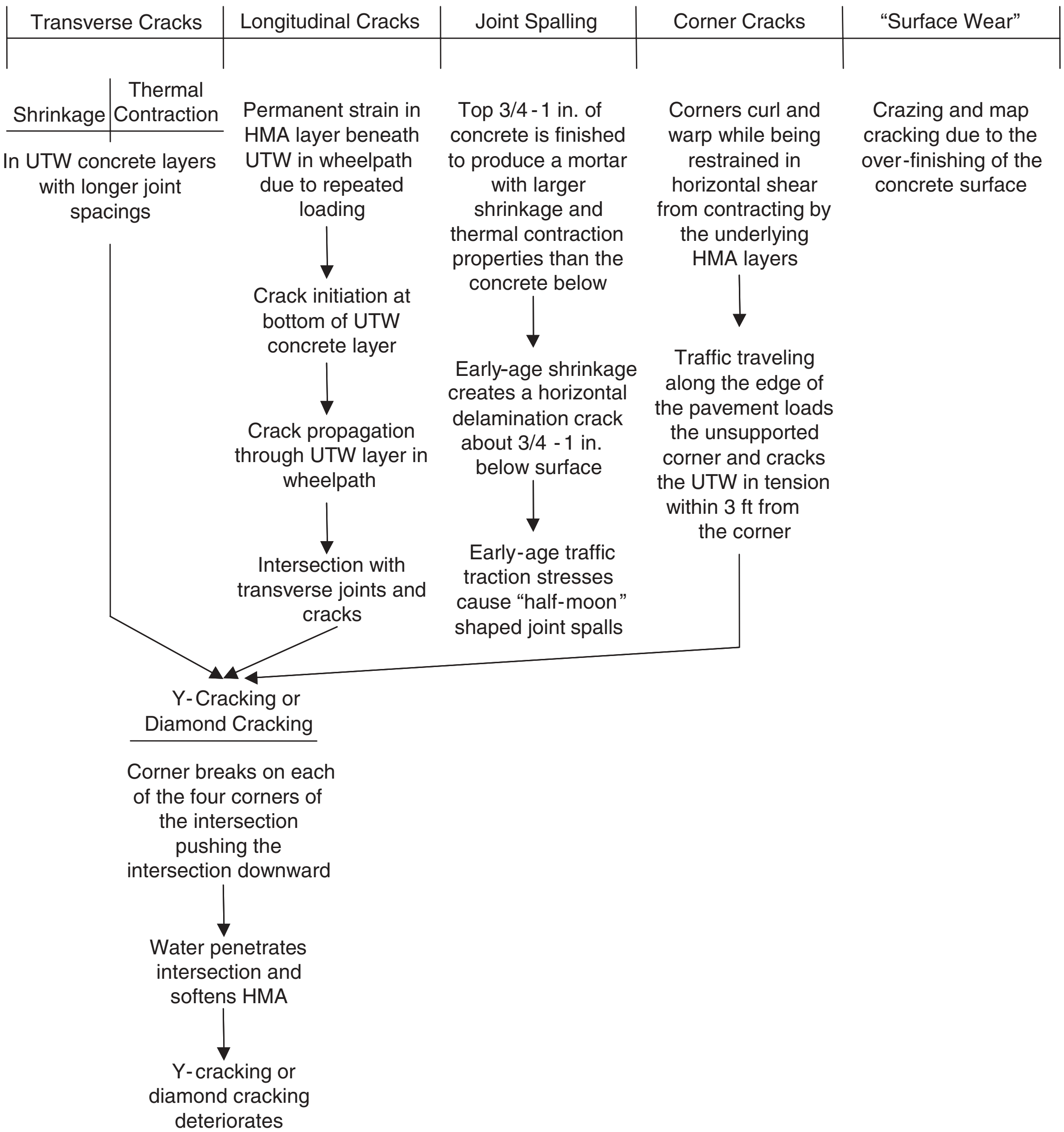

FIGURE 28 Fault tree indicating distress development for UTW (*2*).

- Determine possible alternatives.
- Develop traffic management plans.
- Conduct a cost analysis.
- Select the most appropriate process.
- Execute fieldwork for localized repair, rehabilitation, or replacement activity.

Each of these steps will be discussed in the following sections. The general decision process is described by the flowchart in Figure 29. The major decisions and analyses are included in this flowchart and can help in determining the most appropriate action. In some cases, the most appropriate action will be to do nothing at that time. In other cases, replacing an entire panel may be the best decision. Whether considering the thickness of the whitetopping overlay, the cost analysis, the importance of quickly reopening the facility to traffic, and other factors, the flowchart will lead the evaluation to the recommended rehabilitation activity.

Determine Classification

The first step in this process is to determine the classification of the whitetopping overlay to be rehabilitated—UTW or TWT. The repair of UTW overlays is unique owing to their geometry and degree of bond with the underlying HMA. However, rehabilitation options for TWT will usually be more numerous than for UTW and they will depend on the project specifics. A decision must be made on how to treat it: as a conventional concrete pavement or as being similar to a UTW. These decisions should be made based on the agency's experience and attitudes toward pavement rehabilitation, as well as on engineering principles.

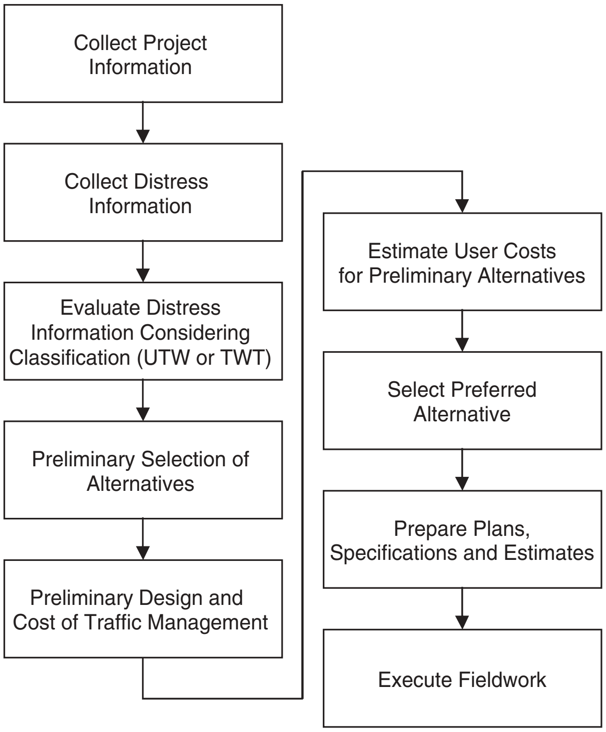

FIGURE 29 Flowchart for evaluating recommended rehabilitation activity (57).

Once the whitetopping classification has been determined, the next step is to evaluate the distresses, their severity, and their effect on the structural and functional performance of the overlay.

Evaluate Distresses

It is important to determine the nature of the distresses observed from the UTW or TWT before moving forward with a predetermined rehabilitation strategy. To make preventive maintenance decisions before a condition becomes critical and warrants expensive measures, it is important to evaluate the condition of the overlay at regular intervals and to make note of actions that could prolong the life of the pavement.

Once distresses have developed in the pavement surface, further evaluation should be conducted to assess the effect of those distresses on the structural and functional performance of the overlay (*1*). For example, it has been found that a crack in a concrete surface will not necessarily impair the performance of the pavement structure (*132*). Furthermore, it is hypothesized that it may be as expensive to seal the crack, with all the associated costs such as traffic control, as it is to wait for the panel to become a performance problem and then to replace the entire panel.

The remainder of this section describes a number of typical distresses that have been observed in UTW and TWT overlays. These distresses include the following:

- Corner cracking,
- Midpanel cracking,
- Shattered slab,
- Joint or crack spalling,
- Joint or crack faulting,
- Surface wear,
- Permanent deformation of support layers,
- Durability, and
- Corner and panel debonding.

Several publications are available to aid in developing a maintenance and rehabilitation strategy (*57,144*). Although many of these publications are intended for conventional PCC pavements, the concepts contained in them can be applied to developing strategies for TWT and some UTW overlays as well.

Corner Cracking

Corner cracking can occur as a result of either wheel loading alone or in combination with another distress. Examples are wheel loading and permanent deformation of layers below the corner or wheel loading and debonding of the whitetopping overlay from the existing HMA layer (*48*). According to survey respondents, this is the most commonly observed distress on UTW, and the second most commonly observed distress on TWT overlays. Figure 30 shows a typical corner crack shortly after developing. Figure 31 shows how a corner crack can further deteriorate under repeated traffic loading.

A corner crack may or may not affect the performance of the overlay, depending on the severity and cause of the crack. If the corner crack results from deformation of the underlying layer, or from debonding, then it may be necessary to repair the crack with a full-depth repair (*145*). If the cracked corner is not faulted or debonded, it could be left alone until it becomes a ride problem or hazardous debris problem. Some agencies may wish to repair any corner crack, given its potential to debond and become a hazard to the driving public.

FIGURE 30 Typical corner crack on a UTW.

FIGURE 31 Deteriorated corner crack on a TWT.

Midpanel Cracking

Midpanel cracking can occur together with other distresses, similar to the situation for corner cracking. Midpanel cracking usually does not warrant repairs, especially in UTW (*146*). However, on some TWT overlays, crack sealing may be an appropriate method of repair. Figures 32 and 33 show typical midpanel cracks on TWT and UTW projects, respectively. Figure 32, provided by the Montana DOT, shows an isolated failure that developed as the result of several factors. These factors were reported as including a poor PCC–HMA bond owing to the presence of dried sawcutting slurry, late sawing, and late curing.

FIGURE 32 Typical midpanel crack on a TWT.

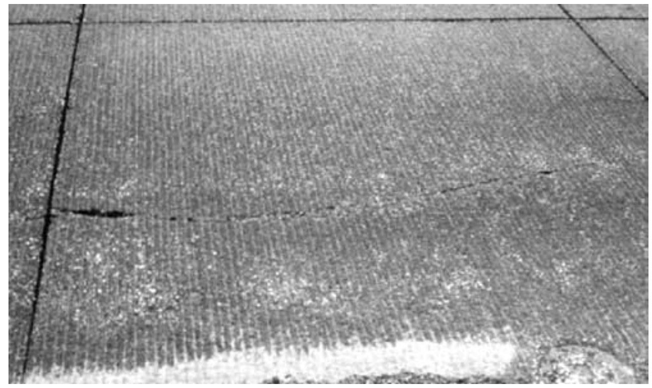

FIGURE 33 Typical midpanel crack on a UTW.

Shattered Slab

As discussed previously, the thinner the whitetopping layer, the more economical it can become to replace an entire panel than to perform routine preventive maintenance. In such cases, the panels are small and thin, and the bond between the concrete and the underlying HMA prevents pieces of concrete from being ejected onto the surface of the pavement. It is important, however, to replace the panel before the first piece of broken concrete is thrown onto the surface, because the effect of traffic is to loosen these pieces after the panel has shattered. Figure 34 shows a slab with several cracks that could be considered to be shattered. Figure 35 shows several slabs that are conclusively shattered, and pieces of concrete

FIGURE 34 Typical shattered slab on a UTW.

FIGURE 35 Severely shattered slabs on a UTW.

attached to some HMAs have been ejected from the pavement structure onto the surface.

Joint or Crack Spalling

A joint or crack can spall under wheel loads or incompressibles between the joint or crack faces. This can indicate a problem such as poor support from one of the underlying layers or a need for joints and cracks to be sealed, depending on the type of whitetopping (*13*). According to survey responses, this distress type is commonly observed, ranking number one for TWT distresses. In UTW, spalled joints or cracks may not cause deterioration of ride quality, but if they do, the panels may be candidates for replacement. Figure 36 shows a severely spalled crack. Spalling of this severity should be repaired as part of the maintenance program.

Joint or Crack Faulting

This type of distress can also be caused by poor support from the HMA or other underlying layer. It can be aggravated by debonding of the PCC from the HMA layer (*48*). This effect has been cited by survey respondents and in the literature. Joints or cracks with severe vertical movement (e.g., faulting) are likely to cause ride quality problems and they should be repaired. As with many distresses, there are multiple ways of addressing faulting, and a cost analysis as well as a repair performance analysis should be performed.

FIGURE 36 Severely spalled crack.

Surface Wear

Surface wear does not necessarily cause ride quality problems, but it should be investigated, for it often is present in more than a few panels. This distress indicates a more systemic problem perhaps related to the construction practices, concrete mix design, or climatic conditions during construction of the overlay. An example of surface wear is shown in Figure 37.

Permanent Deformation of Support Layers

Although it is not usually a cause of distress for PCC pavements, permanent deformation of the support layers has been reported as a potential contributor to whitetopping distress, especially for UTW overlays (*2,48,49*). Permanent deformation of the HMA and other underlying layers can lead to other distresses in the PCC layer, but it is not easily detected unless the PCC distresses are severe. When a slab is replaced or when other rehabilitation is performed that exposes the underlying HMA layer, any permanent deformation in that layer should be addressed before replacing the slab. If this type of distress is suspected, it may be confirmed before construction by trenching or coring. In this way, the rehabilitation activities and construction schedule can be adjusted, as well as the cost estimate for the activity.

Durability

As with surface wear, concrete durability usually indicates a systemic problem, related to the materials used, mix propor-

FIGURE 37 Typical surface wear.

tioning, construction practices, or other conditions during or after overlay construction. Further evaluation should be conducted to estimate the extent of the problem and the potential effects if no action is taken at a given time. Durability problems are very diverse. There are mechanisms by which chemicals and freeze–thaw action can break down the structure of the PCC.

Corner and Panel Debonding

An important characteristic of UTW and most TWT overlays is that they are well bonded to the underlying HMA layer. Sometimes, however, this bond either does not develop during construction or it deteriorates from traffic loading or environmental influences (*2,96*). The result is that the stiffness of the overall pavement structure is compromised and many of the previously discussed distresses can occur. This type of distress can be repaired, but often it is more economical to replace the panel, especially with UTW.

Determine Alternatives

On the basis of the type of distresses observed in the UTW or TWT overlay, a set of possible repair or rehabilitation alternatives can be assembled and ultimately the most appropriate process can be selected. This section describes some of these alternatives.

UTW

The only recommended rehabilitation option for UTW overlays is full panel replacement. Survey respondents reported this as the most commonly used technique. Because UTW panels are small, thin, and easily removed, they do not take much time to replace (*2,54*). UTW overlays are too thin for many types of standard PCC pavement repairs. If a panel is cracked or otherwise distressed, but the ride quality of the pavement is not compromised, the recommended action is to leave the panel in place until a ride quality problem develops (*2*). The panel should be replaced before the first piece of concrete has been ejected from the overlay.

Several problems can affect the ride quality characteristics of a panel in a UTW overlay. As a panel becomes cracked and eventually shattered, it is more likely that pieces of the concrete will be ejected from the pavement structure under the force of vehicular traffic. The presence of moisture increases this likelihood. Stripping can result, whereby moisture breaks down the HMA and thus the bond with the whitetopping concrete. As more cracks develop in the concrete, it is easier for water to infiltrate into the HMA, further accelerating this process. In most cases, as long as the bond between the two layers remains strong, the panel can be left in place. Pavements on a 3% grade or more are reportedly less susceptible to the effects of moisture, because the slope causes water to run off and it is less likely to remain in the pavement structure (*13*).

A UTW panel can typically be replaced very quickly (*2,54*). Figures 38 through 41 illustrate a process undertaken by the Montana DOT for removing and replacing UTW panels, using readily available equipment (*54*). The remove-and-replace-only policy is cost-effective only if the repairs can be made quickly and with minimal impact to traffic. It has been reported that UTW panels can be removed and fresh concrete placed and finished within 2 h. It is recommended that fast-setting concrete be used, which can allow the affected lane to be opened in approximately another 4 h. Thus, the total time required for repairing a panel is only 6 h. Given this time frame, four such panels could be replaced per crew, and pavement can be opened to traffic in only 12 h (*2*).

One suggestion that has been advanced is for agencies to consider developing a fully equipped UTW replacement truck (*2*). A dedicated crew cab pickup truck and trailer can carry all materials required for UTW removal and replacement. Such an arrangement would include a concrete screed, vibrator, jackhammer, compressor, generator, steel mesh for the new panel, and other necessary equipment. The vehicle would also carry four workers who would meet a concrete truck at the site. The trailer also could carry away the old concrete. This configuration has been developed and used by the Colorado–Wyoming chapter of ACPA, and it has been reported to work very well.

Finally, the technique of epoxy injecting has been reported to effectively stabilize loose UTW panels (*147,148*). Although

FIGURE 38 UTW repair step 1—Outline sawing.

FIGURE 39 UTW repair step 2—Removing panels.

effective, it is also expensive and should be considered only if full panel replacement is not an option.

TWT

For TWT overlays, the rehabilitation recommendations begin to resemble those for conventional concrete pavements. This is especially true for TWT thicker than 150 mm (6 in.) or where a bond with the HMA was not intentionally constructed. In these cases, most of the standard concrete pavement repairs can be performed. In addition, as the panel size and thickness increase, those factors influence the decision whether or not to replace the entire panel.

As a result, TWT overlays that are thinner, have short joint spacings, and are bonded to the HMA can be rehabilitated following the guidelines recommended for UTW. Meanwhile, recommended repairs for thicker TWT overlays can include joint resealing, partial-depth repairs (where the area to be repaired is small and shallow), and crack sealing. The

FIGURE 40 UTW repair step 3—Place new PCC.

FIGURE 41 UTW repair step 4—Finish and cure PCC.

prevalence of these repair types was queried in the user survey and the results are summarized in Appendix A. Where a panel has failed, shattered, or has several distresses that should be repaired it may be more cost-effective to replace the entire panel than to repair individual distresses.

Develop Traffic Management Plans

Once the potential maintenance and rehabilitation alternatives have been selected through the engineering analysis, the required traffic management schemes for each method of repair or rehabilitation should be developed. The cost of the traffic management plans for the alternatives must be considered in the cost analysis, described in the next section. There are many publications to aid in developing traffic management plans for pavement maintenance and rehabilitation activities, with the primary source being the *Manual on Uniform Traffic Control Devices* (*149*). Others have been developed that have been tailored for pavement maintenance and rehabilitation (*131,150–152*).

Conduct a Cost Analysis

To develop an effective maintenance and rehabilitation program an agency should conduct a cost analysis, in addition to the engineering feasibility analysis. An appropriate cost analysis should consider both agency and user costs in the evaluation of rehabilitation strategy alternatives (*62*). Agency costs usually consist of the actual construction cost, including labor, materials, and traffic management. However, user costs are more difficult to quantify. The two most prominent and most easily calculated are time motorists spend in traffic that is attributable to work zones and extra fuel expended in work zone traffic (*153,154*). Costs to local businesses, vehicle maintenance and depreciation, and effects of poorly maintained pavement on vehicles and public perception are also important points to consider (*155*). Other costs are more often assigned to "societal costs," or costs borne by society as a whole. They include the effects of air quality owing to work zone traffic, traffic and construction noise, and other intangible costs. Each of these costs, both agency and user, should at least be considered in an evaluation of rehabilitation strategies.

The following sections describe the nature of these costs and give recommendations for evaluating them with respect to rehabilitation strategies. Often, whitetopping repair or rehabilitation strategies that result in lower agency costs can also bring about lower user costs in regard to traffic delay and the impacts associated with delay.

Agency Costs

When an analysis of agency costs is done, only those costs directly paid by the agency should be included. Included are the cost of materials, labor, traffic management, etc. Other items that should be considered, if technically feasible, are rapid-repair materials and techniques.

User Costs

Direct costs to users of a facility are often called user costs. These costs can include tangible items such as extra fuel consumed, oil, and maintenance; they can also pertain to time expended while negotiating a work zone and the associated traffic congestion that often accompanies lane closures. Several studies have been conducted to quantify user costs in various situations and work zone configurations (*64,153,154*).

Other costs that are not as easily quantified include vehicular accidents, public perception, local business costs, and the societal costs mentioned previously. The costs of traffic congestion owing to street and highway maintenance construction, which are borne by society as a whole, include poor air quality and noise.

Select Appropriate Process

Because many UTW and TWT overlays are constructed in urban areas and on highway access ramps, it is imperative that the highway agency allow lane closures judiciously, to minimize the impact to the public. Rapid-repair techniques for panel replacement operation can greatly reduce traffic delays and thus improve public perception about highway maintenance and rehabilitation construction (*155*). Localized repairs can also be done quickly with rapid-repair materials and techniques. Research has been done to identify those materials and techniques that can greatly decrease the time required for construction and the associated lane closures.

Execute Fieldwork for Localized Activities

When engineering and economic analyses show that the best alternative is to perform localized maintenance and rehabilitation, possible alternatives shown in Table 1 are recommended for various distresses (*2*). Each of the recommended rehabilitation alternatives is described in detail in the referenced publications. Although these alternatives are more commonly used for conventional concrete pavements, they may be useful for TWT overlays. It should be stressed that these techniques are not typically used for UTW overlays.

TABLE 1
POTENTIAL DISTRESSES AND RECOMMENDED REHABILITATION ALTERNATIVES FOR TWT

Distresses	Recommended Rehabilitation Alternatives
Corner cracking	Crack sealing (*156*)
	Epoxy injection (*109,147*)
	Cross stitching (*132*)
Midpanel cracking	Crack sealing (*156*)
Shattered slab	Slab replacement (*145,146*)
Joint spalling	Partial-depth repair (*157*)
Joint or crack faulting	Slab stabilization (*148*)
	Load transfer retrofit (*158*)
	Surface grinding (*153,159–161*)
Surface wear (poor skid resistance)	Surface grinding (*153,159–161,**)
Permanent deformation of support layers	Full-depth repair (*145*)
Corner debonding	Epoxy injection (*109,147*)
Panel debonding	Full-depth repair (*145*)
	Slab replacement (*145,146*)

*J. Norris, personal communication (Information on UTW Projects in Tennessee) to T. Ferragut, Sep. 4, 2000,

CHAPTER SEVEN

CONCLUSIONS AND FUTURE RESEARCH NEEDS

As shown from the information reviewed for this synthesis, both thin whitetopping (TWT) and ultra-thin whitetopping (UTW) overlays have been used successfully on hundreds of projects worldwide. When designed and constructed properly, these types of overlays serve as an important option for hot-mix asphalt (HMA) rehabilitation that is readily available to state departments of transportation and other agencies. Although TWT and UTW overlays cannot be used everywhere, they are a viable alternative for roads of light-to-moderate traffic, which still constitute a large percentage of the nation's highways.

Since whitetopping overlays were first used more than 80 years ago, a knowledge base has been developing. Through field trials under a wide range of conditions, best practices have emerged for design and construction of these overlays. This synthesis has identified many of these best practices and it has touched on the state of the art with respect to these overlays.

Whitetopping overlays, including UTW and TWT, have proven to be successful rehabilitation methods when used properly. Although this document has identified numerous practices that can be adopted to ensure success in design, construction, and maintenance, a few of the more important conclusions are as follows:

- The performance of UTW and some TWT overlays are closely correlated with the characteristics of the support layers, especially the HMA layer. If specific and careful consideration is not made toward characterizing the existing pavement system, the whitetopping overlay may be designed or constructed improperly.
- For UTW and TWT overlays to continue to be considered a viable rehabilitation alternative, specifiers and designers should recognize its limitations. As with other portland cement concrete pavements, UTW and TWT overlays have their inherent benefits. However, if they are applied improperly, their reputation as an engineering solution can be tarnished. It needs to be recognized that whitetopping overlays are not a cure-all.
- With respect to whitetopping design, a balance must be struck between accuracy (reliability) and simplicity. Although an overlay design of high accuracy is preferred, the designer must avoid making the product too difficult to use. That is, the designer should be cognizant of the cost and difficulty in collecting information about the existing pavement and balance those findings with the added benefits that would be gained if that information were available. Examples are a lower variability and a factor of safety, leading to a more economical design.
- Although whitetopping overlays have been used for many years, many questions remain about their proper use. This synthesis may serve as a tool to better understand the various issues, although the value of local experience cannot be overstated. Whitetopping projects can be used with increasing success as engineers draw from the experience of the design, construction, and performance of existing overlays in their areas.

Finally, there are some key factors to consider when selecting, designing, and constructing a UTW or TWT project, including

- Distress mode and severity of the existing HMA pavement,
- Stiffness of the existing HMA pavement,
- Proper thickness and joint design for the UTW or TWT,
- Surface preparation of the HMA before overlay (commonly milled and cleaned),
- Fiber reinforcement for UTW and possibly TWT concrete,
- Proper joint sawing depth and timing, and
- Proper curing practices.

From the synthesis survey results, it is clear that there are a number of issues related to UTW and TWT that warrant additional investigation. The following list describes some of the more pressing issues identified during this effort.

- Design and construction standards—It has been asked if thinner concrete overlays should be designed and constructed using the same high-quality standards as for more conventional concrete pavements. If the expected life of the overlay is shorter, can some of these standards be relaxed accordingly? If so, which standards, and to what degree can the quality be reduced without unexpected consequences?
- Joint spacing—Although there have been a number of recent studies to develop mechanistic–empirical models for whitetopping overlays, unanswered questions remain about the optimum joint spacing. Given the numerous competing economic and performance issues, research may be needed to address such questions.

- Preoverlay repair—Whitetopping overlays are commonly used as a rapidly constructed but long-lasting rehabilitation method. With respect to expectations about longevity, questions remain about the effects of preoverlay repairs. The types and quantities of various repairs might be explored with respect to cost first and then the effect on long-term performance of the overlay.
- Bond—Whereas recent studies have revealed the need for adequate bond between the HMA and the whitetopping overlay, little has been done to quantify these effects. More specifically, the relationship between the quality (properties) of the HMA and the bond strength should be explored. Furthermore, the effect on bond from various surface preparation techniques (e.g., milling) should be quantified.
- Rehabilitation—Because many of the UTW and TWT overlays in service are yet to reach their terminal conditions, questions still remain about what techniques are available and cost-effective to rehabilitate or reconstruct these overlays. For example, can a second overlay be used? Can the concrete be milled and replaced? A study that synthesizes and builds on the limited field experience in this area could be beneficial to the industry.
- Performance—Modeling pavement performance under varying traffic, environmental, and other conditions continues to evolve. At the same time, performance data on in-service whitetopping overlays continue to be collected. Research to further the ability to reliably predict UTW and TWT performance could take priority if this overlay type is to continue to be used cost-effectively.

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GLOSSARY OF ACRONYMS

ACI	American Concrete Institute
ACPA	American Concrete Pavement Association
AEA	air entraining agent
ALF	Accelerated Loading Facility
BCO	bonded concrete overlay
CIPR	cold in-place recycling
CRCP	continuously reinforced concrete pavement
CTE	coefficient of thermal expansion
DBI	dowel bar inserter
DCP	dynamic cone penetrometer
ESAL	equivalent (18-kip or 80-kN) single-axle load
FEM	finite-element method (or finite-element model)
FRC	fiber-reinforced concrete
FWD	falling weight deflectometer
GGBFS	ground-granulated blast furnace slag
HIPERPAV	High Performance Concrete Paving Software
HMA	hot-mix asphalt
IPRF	Innovative Pavement Research Foundation
JPCP	jointed plain concrete pavement
JRCP	jointed reinforced concrete pavement
LCC	life-cycle cost
NPV	net present value
PCA	Portland Cement Association
PCC	portland cement concrete
PCI	pavement condition index
PWL	percent within limits
QC/QA	quality control/quality assurance
SCM	supplementary cementitious material
SHA	state highway agency
TFHRC	Turner–Fairbank Highway Research Center
TWT	thin whitetopping
UBOL	unbonded concrete overlay
UTW	ultrathin whitetopping

APPENDIX A

Responding Agency Information

Responding Agency	City	State
Alaska DOT & Public Facilities (PF)	Juneau	AK
Arizona DOT	Phoenix	AZ
Arkansas State Highway and Trans. Dept. (SH&TD)	Little Rock	AR
Colorado DOT	Aurora	CO
Connecticut DOT	Rocky Hill	CT
Delaware DOT	Dover	DE
Florida DOT	Tallahassee	FL
Georgia DOT	Forest Park	GA
Hawaii DOT	Honolulu	HI
Illinois DOT	Springfield	IL
Indiana DOT	West Lafayette	IN
Iowa DOT	Ames	IA
Kansas DOT	Topeka	KS
Louisiana DOTD Louisiana Trans. Res. Center (LTRC)	Baton Rouge	LA
Maine DOT	Augusta	ME
Maryland State Highway Administration (SHA)	Brooklandville	MD
Michigan DOT	Lansing	MI
Minnesota DOT	Maplewood	MN
Mississippi DOT	Jackson	MS
Montana DOT	Helena	MT
Nebraska Department of Roads (DOR)	Omaha	NE
New Hampshire DOT	Concord	NH
New Jersey DOT	Trenton	NJ
New York State DOT	Albany	NY
North Dakota DOT	Bismarck	ND
Ohio DOT	Columbus	OH
Oklahoma DOT	Antlers	OK
Oregon DOT	Eugene	OR
South Carolina DOT	Columbia	SC
South Dakota DOT	Pierre	SD
Texas DOT	Austin	TX
Utah DOT	Salt Lake City	UT
Vermont Agency of Transportation	Montpelier	VT
Virginia DOT Virginia Trans. Research Council (VTRC)	Charlottesville	VA
Washington State DOT	Olympia	WA
Wisconsin DOT	Madison	WI
Wyoming DOT	Cheyenne	WY
ACPA—Illinois Chapter, Inc.	Springfield	IL
ACPA—Missouri/Kansas Chapter	Overland Park	KS
ACPA—NE Chapter	Caldwell	NJ
ACPA—NE Chapter	Buffalo	NY
ACPA—Oklahoma/Arkansas Chapter	Oklahoma City	OK
ACPA—Utah Chapter	Park City	UT
F.A. Rohr Bach, Inc. (Contractor)	Allentown	PA
Ministry of Transportation, British Columbia	Victoria	(Canada) BC
Manitoba Transportation and Government Services	Winnipeg	(Canada) MB
New Brunswick DOT	Fredericton	(Canada) NB
Dept. Works, Srvcs. & Trans., Newfoundland and Labrador	St. John's	(Canada) NL
Ministere des Transports du Quebec	Quebec	(Canada) QC

GENERAL

1. **How many whitetopping overlay projects has your organization been involved in the last year?**
   Average 0.9 projects per respondent.
   Average 2.3 projects per respondent with at least one project.
   Maximum 10 projects.

2. **Of those, approximately what percentage are UTW [less than or equal to 100 mm (4 in.)]?**
   Average 66%.

3. **How many in the last five years?**
   Average 4.2 projects per respondent.
   Average 6.0 projects per respondent with at least one project.
   Maximum 40 projects.

4. **How many in total?**
   Average 5.6 projects per respondent.
   Average 6.6 projects per respondent with at least one project.
   Maximum 50 projects.

   Over what period of time (years)?
   Average 8 years.
   Minimum 1 year.
   Maximum 28 years.

5. **Is there a specific "typical" project that exemplifies current whitetopping practice in your jurisdiction?**
   57% Yes.
   43% No.

6. **If yes to #5, please identify the location, date of construction, and performance of the project. If a report is available with additional details, please provide a citation:**
   - Colorado DOT: SH-83 (Parker Road)—Parker Road has had three thin whitetopping projects over 6 years; one project in Region 6, two in Region 1. The dates of construction were 1997, 1999, and 2002. All three projects were 5.5 to 6.0 in. thick, two with fibers; all have similar traffic and subgrade conditions. To date, two of three have excellent performance and one has very good performance.
   - Delaware DOT: We whitetopped an existing HMA ramp that came off an Interstate to an intersection. Portions of the panels have cracked and spalled due to an adjacent construction project where heavy trucks ran over the corners of the panels.
   - Georgia DOT: Our first UTW was 2.75 in. in an approach to a static scale in a truck weigh station on I-85 in Franklin County. This job was placed in 1993 and has served well. It was placed in an area of severe rutting of the asphalt that was being repaired every 6 months. The whitetopping had some minor repairs over the years due to cracked panels. All panels were 2 ft x 2 ft. This truck weigh station is being upgraded at this time and the UTW was removed in 12/02. We have four other jobs in the southern part of the state, also at intersections where heavy truck traffic was causing rutting and shoving of the asphalt. These have also performed well over the years with little repairs required.
   - Illinois DOT: State Highway System—US Highway 51 and Pleasant Hill Road in Carbondale. This project was constructed in 1998 and unfortunately was removed this summer due to a realignment of US Highway 51 through Carbondale. Performance was very good prior to removal. Local Highway System—County Highway 3 between Louisville and Sailor Springs in Clay County. This project was also constructed in 1998 and performance has been very good to date.
   - Minnesota DOT: TH-30, Amboy. Constructed July 1993. Six 6-in.-thick whitetopping test sections. Very good performance after 9 years of service. Very low volume road, however.
   - Mississippi DOT: US-72 at Hinton Street in Corinth. Constructed in April 2001 with poor to satisfactory performance to date. Surface is too rough (IRI = 4.75 mm/m) and a couple of panel corner breaks will become a maintenance issue. Report available in the TRIS database (AN:00929214 "Construction, Testing, and Preliminary Performance on the Resin Modified Pavement Demonstration Project" Author: R.L. Battey).
   - Montana DOT: Contact Bob Weber or Craig Abernathy for more information.
   - South Carolina DOT: Location—Intersection of SC-707 and US-17, Myrtle Beach. Date of construction: Oct. 2000; Performance—Very good.

- Texas DOT: Intersections of LP250 with Garfield and Modkiff, Midland. Construction completed in the summer of 2001 with excellent performance so far. The only distress observed so far is minor cracking on some panels. Project consisted of removing existing asphalt and base to depth of 9 in., placed 7 in. of a TY B dense-graded mix, milled 1 in., placed 3 in. of UTW. Used quick setting concrete 3,000 psi in 24 h, 3 # of fiber/cy, 5% to 7% air entrainment. Carpet drag or rough broom finish. No technical report has been prepared for the project.
- Virginia DOT (VTRC): Rte 29 NBL approximately 10 mi south of Charlottesville, July 1995. 2000 ft project, 2 lanes, 2-in., 3-in., and 4-in. thick sections. 800 ft of 2-in thick section was badly cracked, began spalling, and had to be removed. Two reports are available: Interim Report: *Evaluation of the Initial Condition of Hydraulic Cement Concrete Overlays on Three Pavements in Virginia*, M.M. Sprinkel and C. Ozyildirim, VTRC 99-IR3, April 1999. Final Report: *Evaluation of Hydraulic Cement Concrete Overlays on Three Pavements in Virginia*, M.M. Sprinkel and C. Ozyildirim, FHWA/VTRC 01-R2, Aug. 2000.
- Wisconsin DOT: State Trunk Highway 82 in Adams County. Constructed in 2001. The performance to date is excellent.
- ACPA—Illinois Chapter, Inc.: Piatt County near Monticello is typical. County job, 5-in. minimum thickness. The UTW projects have been an assortment.
- ACPA—Missouri/Kansas Chapter: Intersection of College and Pflumm in Overland Park, Olathe, and Lenexa, Kansas (corner of all three cities). The project was constructed in 1996 and has performed well. There are a few cracked panels at the curb edge that need repair and a few out in the wheelpath that remain intact due to the fibers and bond. We can get pictures if needed.
- ACPA—NE Chapter: The project is a New York State DOT whitetopping. It was an intersection at Waldon and Central Avenues and was constructed in 2002. The project was unique in that the contractor had to place new asphalt, then mill the depth of 2 in. and overlay the area with 4 to 5 in. of PCC pavement. The performance is adequate; however, there was minor cracking where the concrete was placed over the newly placed asphalt. In areas where there was sufficient existing asphalt, the concrete did not exhibit cracking.
- ACPA—Utah Chapter: Location: SR-198 at Arrowhead Trail, Spanish Fork, Fall of 2002, Performance: Excellent to date.
- F.A. Rohr Bach, Inc. (Contractor): Shortlidge Road, State College, Pennsylvania, July 2001. Project was designed as a 3-in. UTW overlay of a downgrade intersection approach. The project is being monitored. Cracks have developed along the longitudinal edges, but have not had an adverse effect on the performance of the overlay. The previous pavement had terrible washboarding and was milled and overlaid every 3 years. As long as the cracking does not cause further deterioration of the UTW, it would be successful.

General Comments:

- Colorado DOT: CDOT has two urban and one rural thin whitetopping projects along with test sections constructed in 1994 and 1990.
- Georgia DOT: We currently have a project this year in Savannah, with 43,000 yd^2, 4 in. thick, to be placed in intersections on SR-204.
- Illinois DOT: A construction and early performance report is available for these and other projects in Illinois. T.J. Winkelman, *Whitetopping Construction and Early Performance in Illinois*, Illinois DOT, Bureau of Materials and Physical Research Report No. 144, June 2002.
- Maine DOT: Maine DOT is hopefully going to construct a 4 in. ultra-thin whitetopping later this year. The location is a busy intersection in Portland. Estimated quantity is nearly 400 yd^3.
- Michigan DOT: Michigan completed its only whitetopping project to date in 1999. The project involved four test sections: 150 mm with fibers, 150 mm without fibers, 125 mm with fibers, and 75 mm ultra thin. The ultra thin is not a whitetopping, strictly speaking. It was over a composite pavement.
- Minnesota DOT: For more information, see the 2003 TRB paper "Whitetopping and Hot-Mix Asphalt Overlay Treatments for Flexible Pavement: A Minnesota Case History," by T. Burnham and D. Rettner.
- Nebraska DOR: Whitetopping sections of the projects are intermittent and consist of 9-in. PCC over AC. AC thickness depended on how much milling was needed to match the required profile and elevation.
- New York State DOT: NYSDOT has had good success with its UTW projects over the past 5 years.
- Oklahoma DOT: Division II has done two whitetopping projects on US-69 because our asphalt overlays were not lasting very long (4 years usually) and we were looking for something better. These two projects have been done with Division II maintenance of our roads, such as overlays, armor coats, and now whitetopping. Because our maintenance money is limited, these projects are small (first project was 1.5 mi long, second was 1.2 mi long).
- South Carolina DOT: Four-inch UTW with 4-ft joint spacing.
- Texas DOT: Overall, TxDOT is happy with UTW performance.
- Utah DOT: All of our recent whitetopping projects have been UTW.

- Virginia DOT (VTRC): Whitetopping should not be placed at a thickness of 2 in. Thicknesses of 3 and 4 in. are performing well. Based on the project, whitetopping is not practical relative to asphalt at most locations because of the high cost and long lane closure time required for construction and curing.
- ACPA—Illinois Chapter, Inc.: To say there is a general type of project is probably a misstatement. Every one of the projects seems to have its own unique aspects. These are detailed on the attached sheets.
- ACPA—NE Chapter: This project was the first for the Port Authority of New York and New Jersey (PANYNJ) in the state of New Jersey. A larger one has been awarded by PANYNJ at JFK Airport in New York. That project has not started to date.
- ACPA—Utah Chapter: This UTW was part of an asphalt contract. UDOT requested a change order to replace the intersection with UTW. Project was on a 5% grade and exhibited AC ruts greater than 2.5 in. in places. All construction was completed in less than 1 week.

PROJECT SELECTION

7. Rank the following criteria for their influence on the decision-making process for hot-mix asphalt pavement rehabilitation strategies where whitetopping is a viable alternative.
1 = Strongly considered; 2 = Routinely considered; 3 = Occasionally considered;
4 = Rarely considered; 5 = Not/never considered.

Initial cost	Average 1.52 (Rank: 1-tied).
Longevity (life) of “fix”	Average 1.52 (Rank: 1-tied).
Traffic control	Average 1.71 (Rank: 3).
Life-cycle cost	Average 2.44 (Rank: 4).
Improvement to smoothness	Average 2.56 (Rank: 5).
Improvement to texture (safety)	Average 2.64 (Rank: 6).
Geometric design	Average 2.68 (Rank: 7).
Agency experience	Average 2.84 (Rank: 8).
Contractor experience	Average 3.48 (Rank: 9).
Improvement to noise	Average 3.72 (Rank: 10).

8. Do you run life-cycle cost analyses of whitetopping strategies?
27% Yes.
73% No.

9. If yes to #8, what is the basis for unit costs?

Area (e.g., square miles) only	57%.
Volume (e.g., cubic meters)	0%.
Combination of area and volume	43%.

10. Do you consider surface preparation (e.g., milling) prior to placing the overlay as a separate pay item?
85% Yes.
15% No.

Comments on Project Selection:

- Georgia DOT: We core the existing pavement and evaluate the existing asphalt to determine thickness and quality. We mill off 4 in. and try to maintain at least 4 in. of asphalt for support. Requests are normally made by our maintenance department, about where asphalt problems are frequently occurring.
- Michigan DOT: I was not involved in the design phase and therefore cannot answer all of these questions. I have only been involved to record construction information and monitor performance.
- Nebraska DOR: Whitetopping is rarely considered as a rehabilitation strategy.
- New York State DOT: Intersections and ramps that have rutting or shoving as their primary form of distress are primary candidates for UTW.
- Oklahoma DOT: Our project was getting an asphalt overlay about every 4 years and we were looking for something that would last longer. The main problem with this road was rutting caused by all the truck traffic. Our traffic count is 16,200 vpd, with 35% trucks. These concrete whitetopping project costs are about twice as much as our asphalt overlay would cost but if it lasts longer than 8 years then we are financially ahead.
- South Carolina DOT: UTW is generally considered after HMA strategies have been repeatedly unsuccessful.

- Virginia DOT (VTRC): We are not using whitetopping for reasons cited in Question 6. In situations where rutting asphalt is a problem (some intersections and turning lanes), we are constructing a full-depth concrete pavement starting with the base.
- ACPA—Missouri/Kansas Chapter: Most of our UTWs are at intersections where asphalt has historically not held up well (3-year overlay cycles). Many of the cities in the Kansas City area use UTW as a standard maintenance procedure for these intersections despite the remainder of their projects being asphalt. The initial cost of the UTW is substantially higher than an AC overlay (4–6 times), but most of the agencies feel a 12–15 year fix will warrant the added expense.
- ACPA—NE Chapter: This area in Port Newark, New Jersey, has a tremendous amount of heavy truck traffic. This area contains numerous shipper terminals with containers coming and going. The PANYNJ has experienced many asphalt failures at numerous traffic lights in the area.
- ACPA—NE Chapter: The NYSDOT Regional Maintenance Engineer wanted a longer term fix.
- ACPA—Utah Chapter: UTW is a new process in Utah. Whitetopping (>4 in. is not), but nothing has been placed since the late 1980s. Have no idea why. Performance of whitetopping on Interstates throughout Utah has been excellent. Performance of UTW in Utah (oldest ones placed in 1999) has been excellent as well. Comes down to money.
- F.A. Rohr Bach, Inc. (Contractor): Extensive effort needs to be made in determining the thickness of the existing road base, particularly at the lowest point of the UTW, as that is probably a water flow line and has had the least amount of overlay.

DESIGN

11. What techniques are used to evaluate the existing hot-mix asphalt layer for use in whitetopping design?

Visual inspection of condition	90%.
Pavement management data	62%.
Falling weight deflectometer (FWD)	45%.
Rut measurements	76%.
Lab testing of hot-mix asphalt	7%.

Test Method:
- Georgia DOT: Tensile

None 0%.

Other (please elaborate):
- Colorado DOT: Cracking, in-place thickness, condition, etc.
- Montana DOT: Cores.
- South Carolina DOT: Visual inspection of pavement cores.
- Texas DOT: Cores of existing pavement taken to determine depth and quality of various layers of the existing pavement structure.
- Utah DOT: Cores.
- Virginia DOT (VTRC): All answers apply to the 1995 installation.
- ACPA—Illinois Chapter, Inc.: Cores.
- ACPA—NE Chapter: Cores (thickness and subbase evaluation).
- F.A. Rohr Bach, Inc. (Contractor): Cores (thickness).

12. What techniques are used to evaluate the existing support (subgrade/base) layers for use in whitetopping design?

Visual inspection of condition	52%.
Falling weight deflectometer (FWD)	45%.
Rut measurements	24%.
Lab testing of subgrade and/or subbase	17%.

Test Method:
- Colorado DOT: *R*-value and soil classification.
- Maine DOT: Borings to subgrade.
- Nebraska DOR: Moisture and density.

None 14%.

Other (please elaborate):
- Colorado DOT: FWD or a layer analysis might be used for design.
- Delaware DOT: Past experience.
- Louisiana DOTD (LTRC): It is anticipated that in the near future FWD will be used for subbase evaluation and the DCP (dynamic cone penetrometer) will be used for base/subgrade evaluations.
- Montana DOT: Some subsurface investigation (if needed).

- Wisconsin DOT: As-builts.
- ACPA—NE Chapter: History.

13. What pre-overlay repairs are routinely conducted prior to whitetopping overlay placement?

Pothole “filling”	21%.
Subgrade repairs (full-depth patching)	28%.
Crack sealing	10%.
None	38%.

Other (please elaborate)

- Montana DOT: Full-depth PCCP at localized areas.
- Nebraska DOR: Pavement repair would be performed.
- South Carolina DOT: UTW is typically used where the pavement is rutted but not otherwise distressed. Consequently, if pre-overlay repairs are needed, the site is not a suitable candidate for UTW.
- Virginia DOT (VTRC): None needed.

14. What surface preparation method or combination of methods is most commonly used prior to whitetopping overlay placement?

Milling (preceded by pre-overlay repairs)	27%.
Milling (without pre-overlay repairs)	63%.
Sweeping only	33%.
Air blast	37%.
Water blast	30%.
None	3%.

Other (please elaborate):

- Louisiana DOTD (LTRC): We have used the above-mentioned methods. Currently we are just milling and sweeping.
- Montana DOT: Vacuum sweepers.
- New York State DOT: We sand blast clean any surface where residue builds up.
- Oklahoma DOT: We mill existing pavement to eliminate ruts and help control yield. Also, this rough texture helps the concrete to bond to the asphalt and puts the neutral axis of the composite pavement in the asphalt section.
- Virginia DOT (VTRC): Sweeping and spraying water on the milled surface.

15. How is traffic loading characterized for whitetopping overlay design?

Equivalent single-axle loads (ESALs)	59%.
Axle load spectra	0%.
ADT and percent trucks	38%.
Traffic/roadway loading classification	7%.

Other (please elaborate):

- Georgia DOT: Usually high truck traffic areas, low speed.
- ACPA—Missouri/Kansas Chapter: Not usually considered for UTW.

16. How often are fibers specified for the whitetopping concrete?

64% Always.
32% Sometimes.
4% Never.

17. If applicable, what types of fibers have been specified for the whitetopping concrete?

Synthetic monofilament	37%.
Synthetic fibrillated	78%.
Steel	7%.
Synthetic/steel mixture (in same mix)	0%.

Other (please elaborate):

- Maine DOT: Polypropylene fibers.
- Oklahoma DOT: 1.5-in. Fibermesh fibers.
- Texas DOT: Polypropylene fibers.
- Virginia DOT (VTRC): Steel would not be used again because it rusts away in cracks.
- ACPA—Illinois Chapter, Inc.: New Strux 90/40, a Grace structural fiber.

18. What percent (%) of the projects employ "fast-track" (rapid strength) concrete mixtures?

Average 52%.

19. If "fast-track" techniques are used, what criterion is used to open to traffic?

Compressive strength	91%.	
Strength		Average: 2,933 psi.
		Range: 2,000 to 4,000 psi.
Third point flexural strength	5%.	
Strength		Average: 350 (one respondent).
Time from construction (age)	14%.	
Age		Average 53 h.
		Range: 5 h to 7 days.

Other (please elaborate):

- Colorado DOT: Most of the fast-track concrete was accepted using the maturity method based on test slabs and compressive strength.
- Louisiana DOTD (LTRC): New jobs will have a flexural strength requirement.

20. Are climatological factors (e.g., air temperatures) used in the whitetopping design procedure?

8% Yes.
12% Sometimes.
81% No.

If so, how?

- ACPA—NE Chapter: Mix design, maturity, and strength.

21. How deep are the saw cuts made?

Average 30% slab thickness (transverse). (Note: 2 respondents noted "1 in.").
Average 30% slab thickness (longitudinal). (Note: 1 respondent noted "1 in.").

22. Are the joints sealed?

16% Yes.
84% No.

If so, what material is used for sealing?

- Colorado DOT: Silicone joint sealer or preformed joint sealer (often used for fast track).
- Michigan DOT: Hot-pour rubber.
- Nebraska DOR: Hot pour.
- Texas DOT: Class 5 joint sealant at construction joint. Sawcut joints are not sealed.

23. How are transitions made from the whitetopping to the adjacent pavement?

Taper to a thickened concrete section 60%.
Reinforced panels at transition 7%.
Other (please elaborate):

- Colorado DOT: Usually asphalt pavement was overlaid to bring level even with whitetopping.
- Delaware DOT: Hot-mix wedge.
- Georgia DOT: Have placed with and without thickened areas.
- Oklahoma DOT: We start the project at the end of a bridge or concrete pavement and mill the full thickness of the whitetopping project at this location.
- Virginia DOT (VTRC): Whitetopping was an inlay.
- ACPA—Illinois Chapter, Inc.: AC fillet typically. Inlays just butt up to the existing asphalt.
- ACPA—Missouri/Kansas Chapter: Most agencies forget about the thickened edge.

24. What are other design features or concrete-making materials unique to whitetopping overlays (compared to other concrete pavements in your jurisdiction)?

- Michigan DOT: Our ultra-thin section was unsealed with 1 to 1.25 m joint spacing in both directions. We did not use dowel bars at the transverse joints in the regular whitetopping. We did use deformed tie bars in the longitudinal joints, however.
- Mississippi DOT: Plastic spacers are utilized at the crosscuts.
- Montana DOT: Fibers and at times cement types.

- Oklahoma DOT: We use 6 lb of Fibermesh fibers per yard of concrete and use our #67 rock due to the thinner pavements.
- Virginia DOT (VTRC): 2-in.-thick sections done with 0.5-in. maximum size aggregate and 7.5 bags per cubic yard cementitious materials rather than the 1-in. maximum size aggregate and the 6.75 bags per cubic yard used in thicker sections.
- ACPA—Missouri/Kansas Chapter: Most all of our UTWs have been constructed over a weekend so fast-track construction and high early-strength concrete employed.
- ACPA—Oklahoma/Arkansas Chapter: Short joint spacing.
- F.A. Rohr Bach, Inc. (Contractor): Aggregate size is reduced to 3/8 in. material.

25. What existing design procedure is used for whitetopping design (e.g., AASHTO 1993 guide) in your jurisdiction? Please identify the procedure and/or how to obtain a copy.

- Arizona DOT: ACPA guide.
- Colorado DOT: *Guidelines for the Thickness Design of Bonded Whitetopping Pavement in the State of Colorado*, Report No. CDOT-DTD-98-10.
- Delaware DOT: We just went with 4 in. because we had a thick HMA pavement beneath the whitetopping.
- Iowa DOT: AASHTO 1993, ACPA.
- Illinois DOT: ACPA, *Whitetopping—State of the Practice*.
- Louisiana DOTD (LTRC): FHWA-IF-02-045.
- Maine DOT: None has been developed yet.
- Michigan DOT: AASHTO 1993 guide.
- Mississippi DOT: ACPA method.
- Montana DOT: Seat-of-the-pants method.
- Nebraska DOR: AASHTO.
- South Carolina DOT: Thickness by policy only. Existing pavement must have at least 4 in. of asphalt after milling to be eligible for UTW.
- Texas DOT: *Whitetopping—State of the Practice*, Engineering Bulletin EB210.02P, produced by the ACPA.
- Utah DOT: ACPA guidelines.
- Virginia DOT (VTRC): None.
- Wisconsin DOT: We have been using PCA's guidelines for design. We have also been using ISLAB 2000 for stress evaluation.
- ACPA—Illinois Chapter, Inc.: We've used AASHTO 1993 guide. We have also used variations of the spreadsheets upon which the ACPA design procedure is based. I've also used adaptations of the work done by Scott Tarr, CTL, for Colorado and reported at the 2000 TRB annual meeting.
- ACPA—Missouri/Kansas Chapter: Usually no design procedure. Most agencies just select a thickness between 3 and 4 in.
- ACPA—NE Chapter: ACPA Win Pas and New York State Specification for HMA Overlays Utilizing PCC. Obtain a copy from NYSDOT website under Engineering Instructions/Specifications.
- ACPA—Oklahoma/Arkansas Chapter: Win Pas (AASHTO 1993 based).
- ACPA—Utah Chapter: We use ACPA UTW design calculator. Found on www.pavement.com.

26. What procedure is used for determining the joint spacing for whitetopping overlays?

- Arizona DOT: ACPA/PCA guide.
- Colorado DOT: Based on PCC pavement thickness.
- Delaware DOT: ACPA recommendations—whatever the depth of placement (in feet).
- Iowa DOT: Traditional rule of thumb; i.e., 2 in. thick use 2 ft x 2 ft, 3 in. thick use 3 ft x 3 ft. Also use experience from other projects like IA 21. The DOT has only had a couple projects.
- Illinois DOT: The joint spacing (transverse and longitudinal) in feet is generally one to one and a half times the overlay thickness in inches. Example: Overlay thickness = 3.0 in., joint spacing = 3.0 to 4.5 ft. However, we do try to avoid placing a longitudinal joint in the wheelpath or area of high stress. Also, we place full-depth joints over major cracks of the underlying pavement and at the boundaries of any patches that are placed in the underlying pavement. (These full-depth joints are sealed with a hot-poured joint sealant.)
- Louisiana DOTD (LTRC): FHWA-IF-02-045.
- Maine DOT: As per ACPA . . . 1 ft per inch of thickness.
- Michigan DOT: Recommendations from ACPA.
- Mississippi DOT: Early projects the same spacing, in feet as inches of thickness, was used (i.e., 4 in. thick = 4 ft x 4 ft panels). However, currently evolving to a spacing based on Minnesota DOT research that keeps the longitudinal joint out of the wheelpath.
- Montana DOT: Current recommended whitetopping procedures, from ACPA and FWHA.

- Nebraska DOR: Whitetopping has been equal to or greater than 8 in. deep so joint spacing is the same as other pavement.
- New York State DOT: Design guidance for UTW projects can be found in Engineering Instructions (EI) 01-008. All EIs can be found in the department's website under Publications.
- Oklahoma DOT: Our joint spacing is 1 ft for every inch of thickness.
- South Carolina DOT: Policy, previous experience.
- Texas DOT: Joint spacing in feet equal depth of slab in inches.
- Utah DOT: 12 times thickness (inches).
- Virginia DOT (VTRC): Spacing is 12 times the thickness rounded to a number that divides evenly into the lane width.
- ACPA—Illinois Chapter, Inc.: Seat of the pants and experience.
- ACPA—Missouri/Kansas Chapter: Usually the general rule of 12 times the thickness is used in conjunction with lane widths.
- ACPA—NE Chapter: 1 to 1.5 times thickness.
- ACPA—NE Chapter: ACPA guidelines and NYSDOT Specifications for HMA Overlays Utilizing PCC. Typically the thickness will determine the joint spacing.
- ACPA—Oklahoma/Arkansas Chapter: 4 ft to 6 ft for 4-in. UTW, pavements of 4 in. same as normal pavements.
- ACPA—Utah Chapter: Not a procedure. Go off recommended guidelines from ACPA.
- F.A. Rohr Bach, Inc. (Contractor): 3 ft centers rule of thumb.

27. If you have been involved in developing or evaluating a new design or analysis procedure for whitetopping, please provide details on how to obtain additional information.

- Arizona DOT: ACPA/PCA.
- Georgia DOT: Typically use #7 (1/2 in.) max size stone.
- New York State DOT: There are no plans to revise the current design and construction procedures.
- ACPA—Illinois Chapter, Inc.: Not been involved, but would sure like to try something more rational. Currently investigating ISLAB 2000 adaptations.

Comments on Design:

- ACPA—Illinois Chapter, Inc.: Do not have a good procedure for working on top of mixed surfaces. We have several that have combinations of exposed concrete as a direct or partially bonded system combined with UTW. Seems to work okay so far, but has me jumpy based on past experience.
- ACPA—Missouri/Kansas Chapter: 3 to 4 in. appears to be a good thickness for most applications.

CONSTRUCTION

28. If fibers are introduced into the concrete mixture, how is this done?

Pre-mixing with dry constituents	8%.
Adding to wet mixture via bags/bundles	68%.
Adding to wet mixture via loose fibers	16%.

Other (please elaborate):

- Iowa DOT: Blown into drum, wet mixture.
- Michigan DOT: Not known how the contractor accomplished this.
- Montana DOT: Added to empty drum with small amount of water, and then add the dry constituents.
- South Carolina DOT: Method left to concrete producer.
- ACPA—Missouri/Kansas Chapter: Have had problems with fibers balls.

29. Is batching or transportation of concrete for whitetopping overlays done differently than for other concrete paving? If so, please explain how it is different.

18 Respondents answered "No."

- Maine DOT: Batching will be done as our concrete bridges are done.
- South Carolina DOT: UTW is brought in agitator/mixer trucks; dump trucks not allowed.
- Utah DOT: Typically UTW projects are serviced by traditional ready-mix operations, whereas most conventional PCCP projects are serviced by portable pre-mix plants and non-agitating haul trucks.
- ACPA—Illinois Chapter, Inc.: Not done differently so far. We've used central mix for the big jobs and transit for the small ones.
- F.A. Rohr Bach, Inc. (Contractor): Batch loads are smaller (6 to 7 cy) to allow for quick discharge to ensure that slump is not lost.

30. Is placing the concrete for whitetopping overlays done differently than for other concrete paving? If so, please explain how it is different.

12 Respondents answered "No."

- Georgia DOT: Use of small vibratory screeds.
- Iowa DOT: No belt placer.
- Illinois DOT: Standard concrete paving is performed with a mechanical concrete placer followed by a mechanical slip-form concrete paving machine. These machines use a string line to achieve grade control and smoothness. The main-line whitetopping (thin) that we have done was placed in this fashion. However, all of our intersection projects (ultra-thin) have used 2 × 4 lumber as formwork and a vibratory screed for finishing. The concrete is manually placed in front of the vibrator screed.
- Maine DOT: Maine has no new concrete pavements. This will likely be laser screeded or vibratory screeded.
- Michigan DOT: The only difference is that the existing surface was pre-wet prior to concrete placement.
- Mississippi DOT: Usually a paver is not used and unfortunately bull floats are utilized for the surface finish, thus leading to a poor ride quality on most projects.
- New York State DOT: Typically, because of room constraints smaller paving equipment is utilized to place the PCC.
- Texas DOT: Our limited experience is that a different type of screed is used for the UTW. Other than that, the paving operation is very similar to concrete pavement.
- Utah DOT: UTW lends itself to hand placements rather than slip-form operations.
- Virginia DOT (VTRC): Whitetopping is done one lane at a time. Full-depth pavements are often constructed over multiple lanes.
- ACPA—NE Chapter: Yes, we have a couple of different processes for placement. One contractor has a Zero Clearance paver for slipforming whitetopping; others have used roller screed and air "truss" screeds.
- F.A. Rohr Bach, Inc. (Contractor): Much more hand screed to create swale lines. Much more manpower required on a per cubic yard basis.

31. Is finishing and curing the concrete for whitetopping overlays done differently than for other concrete paving? If so, please explain how it is different.

14 Respondents answered "No."

- Georgia DOT: Broom finish in lieu of tining. Same curing.
- Maine DOT: Only a curing compound will be specified on this project.
- Montana DOT: Adding more curing compound (double dose).
- ACPA—Illinois Chapter, Inc.: Sometimes will insist on double coat of curing compound on the thinner sections.
- ACPA—Missouri/Kansas Chapter: Special attention is paid to the timeliness of curing and the application is usually increased to 1.5 to 2 times.
- ACPA—NE Chapter: We specify a double coverage of curing.
- ACPA—Oklahoma/Arkansas Chapter: Double application rate of curing compound.
- ACPA—Utah Chapter: Apply the curing compound at twice the rate of normal paving and get it on rather timely. Sawing in a timely manner.
- F.A. Rohr Bach, Inc. (Contractor): Time is accelerated. Immediately after strike-off it is finished and immediately it is spray cured. Poly covers/wetting is done after sawcutting.

32. Is saw cutting the concrete for whitetopping overlays done differently than for other concrete paving? If so, please explain how it is different.

6 Respondents answered "No."

- Colorado DOT: More often have had to use Soff-Cut saws to prevent premature crack formation, particularly in fast-track sections.
- Delaware DOT: Yes—just earlier.
- Georgia DOT: Use of lightweight saws to make sure joints are sawed before cracking can occur.
- Illinois DOT: Yes. Early entry (lightweight) saws are used to cut the joints as soon as the concrete will support the weight of the saw and operator. Care is taken to avoid premature sawing and spalling of the joints.
- Michigan DOT: The joints widths were 3 mm using one-stage sawcutting. Typically, we use two-stage sawcutting (relief cut followed by a final width cut).
- Montana DOT: No, using early entry saws.
- New York State DOT: We only do a single stage sawcut (T/4) for both longitudinal and transverse sawcuts.
- Oklahoma DOT: We saw more joints and do not seal them.
- South Carolina DOT: Early entry saws required.
- Texas DOT: The sawcutting is done as soon as possible without damaging the surface.

- Utah DOT: Sometimes the contractor chooses early entry saws for UTW.
- Virginia DOT (VTRC): Yes, whitetopping gets 1-in. soft cut; full-depth pavement gets hard cut approximately one-third the thickness of the pavement.
- Wisconsin DOT: Yes on whitetopping we are using a soft cut saw.
- ACPA—Illinois Chapter, Inc.: UTW is cut using early entry saws. Most of the other conventional whitetopping has used regular saws.
- ACPA—Missouri/Kansas Chapter: Usually early entry saws are used, and a lot of them.
- ACPA—Oklahoma/Arkansas Chapter: More saws due to smaller sawing window on UTW. Early entry saws are contractor's option.
- ACPA—Utah Chapter: Just done a bit quicker with higher cementitious mixes.
- F.A. Rohr Bach, Inc. (Contractor): Early cut saws are used instead of traditional wet cut saws.

33. Is quality control of the concrete used in the whitetopping overlays different than for other concrete paving? If so, please explain how it is different.

20 Respondents answered "No."

- Louisiana DOTD (LTRC): No, not at this time. Future jobs will involve flexural strength testing for mix approval in the preconstruction phase of the job.
- Maine DOT: Quality control will be increased due to the experimental nature and critical location.
- South Carolina DOT: PCC controlled by compressive rather than flexural strength.
- F.A. Rohr Bach, Inc. (Contractor): Much more attention has to be given to every aspect of the application. Whitetopping can be very challenging.

34. If you have a standard specification or special provision for whitetopping construction, please attach a copy or describe how to obtain (e.g., Internet address).

- Arizona DOT: No standard specification.
- Colorado DOT: [Online]. http://www.dot.state.co.us/DesignSupport/Construction/1999SSP/412pccp.doc.
- Delaware DOT: Contact Specification Engineer—George Nagase (302-760-2252).
- Illinois DOT: Yes, attached are two special provisions for whitetopping overlays. One is typical of a mainline application and the other is typical of an intersection application (available from synthesis author if requested).
- Louisiana DOTD (LTRC): See attachment and Please Note! This is a draft copy and not finalized but indicates the direction LA DOTD is going.
- Maine DOT: See attached (available from synthesis author if requested).
- Michigan DOT: We do not have a standard specification yet.
- Mississippi DOT: Yes, will attach (available from synthesis author if requested).
- Montana DOT: Current industry standard (ACPA/FHWA).
- Nebraska DOR: None.
- New York State DOT: We have a special specification that we used. It can be found on the website with the EI.
- South Carolina DOT: Copy attached (available from synthesis author if requested).
- Texas DOT: We are in the process of developing a statewide special specification for UTW.
- Virginia DOT (VTRC): No.
- Wisconsin DOT: No.
- ACPA—Illinois Chapter, Inc.: Will attach (available from synthesis author if requested).
- ACPA—Missouri/Kansas Chapter: We have a new APWA Metro specification for Kansas City, which includes UTW. Both MoDOT and KDOT also have specifications.
- ACPA—NE Chapter: 18502.0601 M—Thin Portland Cement Concrete (PCC) Overlay of Hot Mix Asphalt (HMA) Surfaced Pavement, Unprofilographed. 18502.0602 M—Thin Portland Cement Concrete (PCC) Overlay of Hot Mix Asphalt (HMA) Surfaced Pavement, Unprofilographed, High Early Strength Mix [Online]. http://dotweb1.dot.state.ny.us/specs/wpdindex18.html.
- ACPA—Utah Chapter: UDOT is currently revising their UTW specification. You can try (if available) [Online]. www.udot.utah.gov.

Comments on Construction:

- Colorado DOT: Construction has been similar to normal concrete sections.
- New York State DOT: See the EI.
- Virginia DOT (VTRC): No problems.

- F.A. Rohr Bach, Inc. (Contractor): Opening to traffic is an issue. Specifications must clarify the method used to determine strength. I do not believe traditional cylinders are appropriate as a measure of field cure/opening to traffic. Maturity testing should be considered on UTW projects.

PERFORMANCE, REPAIR, AND REHABILITATION

35. For each distress type, please identify how often it is encountered on whitetopping overlays that you have observed: 1 = Commonly observed; 2 = Occasionally observed; 3 = Rarely observed; 4 = Never observed.

	UTW [≤100 mm (4 in.)]	TWT [>100 <200 mm (>4 <8 in.)]
Corner cracking—wheelpath only	Avg. 2.10 (Rank: 1)	Avg. 3.25 (Rank: 2)
Corner cracking—not in wheelpath	Avg. 2.65 (Rank: 2)	Avg. 3.50 (Rank: 5)
Transverse cracking—wheelpath only	Avg. 2.85 (Rank: 5)	Avg. 3.58 (Rank: 6)
Transverse cracking—full lane width	Avg. 3.00 (Rank: 7-tie)	Avg. 3.42 (Rank: 3-tie)
Longitudinal cracking—wheelpath only	Avg. 2.80 (Rank: 4)	Avg. 3.62 (Rank: 7)
Longitudinal cracking—not in wheelpath	Avg. 2.90 (Rank: 6)	Avg. 3.42 (Rank: 3-tie)
Shattered slab—wheelpath only	Avg. 3.00 (Rank: 7-tie)	Avg. 3.92 (Rank: 10-tie)
Shattered slab—not in wheelpath	Avg. 3.20 (Rank: 9)	Avg. 3.92 (Rank: 10-tie)
Joint spalling	Avg. 2.70 (Rank: 3)	Avg. 3.08 (Rank: 1)
Polishing of aggregates	Avg. 3.80 (Rank: 11)	Avg. 3.83 (Rank: 8-tie)
Rapid deterioration of smoothness	Avg. 3.40 (Rank: 10)	Avg. 3.83 (Rank: 8-tie)

Other (please elaborate)—UTW:
- Texas DOT: 3.
- Virginia DOT (VTRC): Spalling of 2-in. thick sections.
- ACPA—NE Chapter: 2—Circular cracks, four panels.
- F.A. Rohr Bach, Inc. (Contractor): 2—faulting due to thin paving subsupport.

Other (please elaborate)—TWT:
- Colorado DOT: One 1997 project has some longitudinal cracking of a construction problem. Most of the other projects are too young to show much distress. The test sections built in 1990 are showing their age, and some will have to be addressed soon.
- Virginia DOT (VTRC): Not used.
- ACPA—NE Chapter: 2—Circular cracks, four panels.

**36. If applicable to your experience, please identify for each repair technique how often it is employed (when needed) on existing whitetopping overlays:
1 = Commonly used for repair; 2 = Occasionally used for repair;
3 = Rarely used for repair; 4 = Never used for repair.**

	UTW [≤100 mm (4 in.)]	TWT [>100 <200 mm (>4 <8 in.)]
Full slab replacement	Avg. 3.40 (Rank: 1)	Avg. 3.20 (Rank: 1)
Partial slab replacement	Avg. 3.64 (Rank: 3)	Avg. 3.56 (Rank: 2)
Crack sealing	Avg. 3.86 (Rank: 5)	Avg. 3.70 (Rank: 4)
Joint (re-)sealing	Avg. 3.79 (Rank: 4)	Avg. 3.60 (Rank: 3)
Spall repair	Avg. 3.57 (Rank: 2)	Avg. 3.89 (Rank: 5-tie)
Dowel bar retrofit	Avg. 4.00 (Rank: 7-tie)	Avg. 3.89 (Rank: 5-tie)
Tie bar retrofit	Avg. 4.00 (Rank: 7-tie)	Avg. 3.89 (Rank: 5-tie)
Diamond grinding	Avg. 3.93 (Rank: 6)	Avg. 3.89 (Rank: 5-tie)

Other (please elaborate)—UTW:
- Louisiana DOTD (LTRC): 4.
- Texas DOT: 3.

- Virginia DOT (VTRC): 4.
- ACPA—NE Chapter: 4.

Other (please elaborate)—TWT:

- Colorado DOT: Most of CDOT's projects are young, so distresses are minimal at this time.
- Virginia DOT (VTRC): None used.
- ACPA—NE Chapter: 4.

37. When a whitetopping overlay has/does reach the end of its life, what options have been/will be considered? 1 = Only option considered; 2 = Strongly considered; 3 = Possibly considered; 4 = Rarely considered; 5 = Not/never considered.

	UTW [≤ 100 mm (4 in.)]	TWT [>100 <200 mm (>4 <8 in.)]
Remove and replace with new whitetopping	Avg. 3.36 (Rank: 2)	Avg. 3.11 (Rank: 2)
Remove and replace with other than whitetopping	Avg. 3.00 (Rank: 1)	Avg. 3.33 (Rank: 3)
Overlay old whitetopping	Avg. 3.43 (Rank: 3)	Avg. 3.00 (Rank: 1)
Break/seat/overlay old whitetopping	Avg. 4.29 (Rank: 4)	Avg. 4.13 (Rank: 4)

Other (please elaborate)—UTW:

- Montana DOT: Too soon to tell.
- New York State DOT: Reconstruct.
- Texas DOT: 4.
- Virginia DOT (VTRC): 5.
- ACPA—Missouri/Kansas Chapter: Have not reached end yet.
- ACPA—NE Chapter: 2—Reconstruct.
- ACPA—Oklahoma/Arkansas Chapter: If DOT and pavement conditions allow, bonded overlay.
- F.A. Rohr Bach, Inc. (Contractor): 2—Full-depth reconstruction.
- Montana DOT: Too soon to tell.

Other (please elaborate)—TWT:

- Montana DOT: Too soon to tell; undecided.
- Virginia DOT (VTRC): Not used.
- ACPA—NE Chapter: Diamond grind.
- ACPA—NE Chapter: 2—Reconstruct.

38. Is there a pre-defined schedule for the various repair and rehabilitation activities listed here? If so, please describe the process and/or identify how a copy might be obtained.

15 Respondents answered "No."

- ACPA—Illinois Chapter, Inc.: Typically in terms of cracked slabs, which is actually too conservative. Shattered slabs would make more sense since a cracked slab frequently remains keyed in or sometimes bonded but with intermediate random cracks. Side note: In several cases, I have seen bonded overlays that have delaminated that will still provide several years of performance, although DOT will rate them as failed panels since they have cracked. Need a more realistic definition of failure.
- ACPA—Missouri/Kansas Chapter: We have not really done any repairs to our UTWs. Most have some cracking, but it appears to be remaining tight and has not affected roughness or performance yet.

39. How is performance of a whitetopping overlay defined? Are there methods in place to measure or evaluate the performance? If so, please describe.

- Arizona DOT: Projects still under design. Construction later this year.
- Colorado DOT: The Pavement Management Annual Survey of pavement condition is one of the main tools for evaluation at this time.
- Delaware DOT: Not available—visual.
- Georgia DOT: Service life.
- Iowa DOT: By amount of cracking and debonding. No formal method to measure performance.

- Illinois DOT: Visual distress surveys performed on an annual basis. The amount of cracking and distress is tracked over the course of the project.
- Louisiana DOTD (LTRC): No.
- Maine DOT: Reduction of pavement rutting.
- Michigan DOT: Yearly visual evaluation for distress, yearly FWD testing, yearly smoothness testing.
- Minnesota DOT: Visual distress (cracking) and ride smoothness.
- Mississippi DOT: MDOT Pavement Management Distress Criteria based on the SHRP LTPP Distress Identification Manual.
- Montana DOT: Crack mapping, delamination/debonding, visual only.
- Nebraska DOR: Performance is measured the same as other rigid pavement.
- New York State DOT: All of the UTW projects are performing well. Rutting and shoving have been eliminated and the traveling public is now assured of a smooth, long lasting ramp or intersection.
- South Carolina DOT: Periodic visual inspection is conducted on the projects completed to date.
- Texas DOT: The amount of time that it performs under traffic with no repairs needed. We have no formal methods in place to measure or evaluate the road other than visual observation.
- Virginia DOT (VTRC): Good, means it has not spalled away.
- Wisconsin DOT: Performance is defined by our Pavement Management System.
- ACPA—Illinois Chapter, Inc.: Not one of which I am aware.
- ACPA—Missouri/Kansas Chapter: Typical methods, visual survey plus ride quality.
- ACPA—NE Chapter: Just standard pavement management practices. Typically a windshield survey followed by FWD if needed.
- ACPA—Oklahoma/Arkansas Chapter: If it lasts a year or longer without severe rutting/shoving then we have been successful. No methods in place to evaluate performance other than visual observation.
- ACPA—Utah Chapter: UDOT has defined as such.

40. Is there a method in place to measure or evaluate the remaining life of a whitetopping overlay? If so, please describe:

13 Respondents answered “No.”

- Arizona DOT: Visual field review and cores.
- Colorado DOT: The pavement management remaining service life curves.
- Maine DOT: Visual inspection.
- Michigan DOT: Michigan has remaining service life using distress data from our Pavement Management System. This typically requires historical data, however. We have no historical data on whitetopping, so my answer would be “no” for our one project.
- Nebraska DOR: Same as other rigid pavement.
- ACPA—Illinois Chapter, Inc.: Full-depth patching of the sections of UTW for which there were problems.

Comments on Performance, Repair, and Rehabilitation:

- Georgia DOT: Our UTW has performed well with minimal repairs. Need to be sure when milling that thin “scab” layers of asphalt are removed and that the underlying layer is not prone to stripping, since our joints are not sealed.
- Michigan DOT: No repairs have been done on our whitetopping project (Question 36) except for a situation where two panels in the UTW were removed to repair a water main leak. The panels were removed full depth and replaced with concrete.
- Mississippi DOT: Most problems in UTW in Mississippi have stemmed from stripping in the underlying HMA, resulting in a loss of bond with the concrete and ultimately failure of the UTW.
- Nebraska DOR: The asphalt layer appears to trap moisture and increase aggregate–alkali reactivity.
- Oklahoma DOT: Our first whitetopping project is 18 months old with no repairs or failures to date. Our newest whitetopping project was completed in December 2002 with no repairs or failures to date.
- South Carolina DOT: The greatest problem we have had with UTW has been the nonuniform nature of pavement thickness in urban settings where the need for UTW is highest. Consequently, despite extensive preconstruction coring, we have encountered areas of low asphalt thickness. These areas do not provide adequate support for the UTW and rapid deterioration is observed. Because of this difficulty, our agency will be using UTW only for special situations and not as a routine procedure.
- ACPA—Missouri/Kansas Chapter: Again, have not really had to do much UTW repair to date. Probably will be coming soon.
- ACPA—Oklahoma/Arkansas Chapter: Out first UTW was a 4 in. intersection overlay. Will be 5 years old this summer and has performed remarkably. Only a few minor distresses observed. Has been in place long enough that engineers/

owners convinced of long-term performance. Expect more to follow soon. Our second attempt is a UTW/TWT hybrid: 4 in. inside lane and 6 in. outside lane. Has performed so well in first year that owner has already produced second identical project.

- ACPA—Utah Chapter: Again, Utah is relatively inexperienced with UTW. Our oldest projects were built in 1999 and are still performing excellently. UTW has been used only in intersection applications in Utah. Whitetopping (difficult to find the information and everyone seems to have forgotten about it) is performing well. For some reason our state does not maintain PCC pavement and just lets them go. The one whitetopping (10.5 in.) I am aware of has exhibited corner breaks and exhibits faulting. UDOT has programmed in to rehab this pavement in the next 5 years.

APPENDIX B

Detailed Case Studies

Currently, a number of states have thin whitetopping (TWT) and ultra-thin whitetopping (UTW) projects that are significant data sources for the industry. These include projects in Minnesota, Colorado, Iowa, Tennessee, Georgia, Missouri, and Virginia. The following sections outline project details of sites from each of these states.

MINNESOTA

The Mn/ROAD Project was constructed in 1994 to provide a full-scale testing facility for the study of climate, materials, and traffic interactions (*50–52*). A total of 40 portland cement concrete (PCC), hot-mix asphalt (HMA), and aggregate surfaced roads are subjected to low- and high-traffic volumes. In October 1997, the Minnesota Department of Transportation (MnDOT) constructed six trial whitetopping sections that are subject to Interstate traffic conditions. These TWT and UTW sections are reinforced with fibers based on the assumption that the fibers can hold the crack widths tight. Typically, whitetopping is used on low-volume roads and at deteriorating intersections, so this test simulates the effects of accelerated loading and investigates the viability of UTW as a rehabilitation technique on highly trafficked roads. In addition, the Minnesota climate is one of the more severe environments that a whitetopping overlay will have to endure. Temperatures can vary greatly there, and the Minnesota spring frost can result in extensive damage to the pavement by traffic.

This site is of interest because it has been heavily instrumented with dynamic, static, moisture, and temperature sensors. In addition, data from these sensors can be used to assess the bond between the UTW and the HMA layers. Bond is believed to be one of the key performance quantifiers of the UTW. If the bond is strong, the two layers behave in a monolithic manner to reduce load-related stresses. If the bond is broken, then the layers act independently, and the benefits of UTW are reduced significantly.

Mn/ROAD Design

As Table B1 illustrates, six sections of fiber-reinforced whitetopping overlay were constructed on an existing 340 mm (13.5 in.) HMA pavement on Minnesota I-94. Traffic on the Interstate is approximately 1 million equivalent single-axle loads (ESALs) per year. To reduce the effects of curling, the whitetopping panels are cut into 1.2 × 1.2, 1.5 × 1.8, and 3.0 × 3.6 m (4 × 4, 5 × 6, and 10 × 12 ft) sections. The joints were sealed as a precaution against Minnesota's temperature extremes and freeze–thaw cycles. Under traffic loading, these shorter panels deflect downward instead of bending, as typical 4.5-m (15-ft) jointed PCC pavements do. The existing HMA pavement contains a Pen 120/150 binder type.

Whitetopping Instrumentation

The test section is subdivided into six cells with various thicknesses, joint patterns, and types of fibers, as indicated in Table B1. Each cell is instrumented with dynamic and static strain, temperature, and moisture sensors. This allows for the measurement of both static and dynamic pavement response under various applied and environmental loading conditions. The hourly strains produced by environmental and applied loads will be recorded for these whitetopping designs.

Distress Survey

The first transverse cracks and corner breaks were found in June 1997. Most of the corner breaks occurred between March and June 1997, and are believed to be the result of high negative temperature gradients (measured in May 1997). When the corners are subjected to a negative temperature gradient, the corners of the whitetopping curl upward, and a gap forms between the pavement support and the overlay slab. Traffic loading subsequently causes the cracks to form as a result of this cantilever effect. It is believed that the cracks remain tight owing to the bridging of the synthetic fibers.

Transverse cracks were found in the whitetopping in January 1999 and are believed to be attributable to the reflection cracking mechanism. Seventy percent of the total number of cracks in the whitetopping is believed to be the result of reflection cracking. Most of the cracks formed in the 75-mm (3-in.) whitetopping overlay, and there are no cracks in the thicker 150-mm (6-in.) sections. However, until the overlay has been subjected to additional loading, it is too early to draw conclusions as to the optimal mix and design.

Results

The bond between the whitetopping and the HMA is believed to be one of the critical properties of the whitetopping system. If the bond is high, then the neutral axis of the PCC–HMA pavement system is lowered and the pavement acts as a monolithic system. If the bond is broken, then the overlay and underlying pavement deform separately, and the life of the overlay will be shortened. To quantify the bond of the overlay to the existing pavement, strain gauges were installed at the top and bottom of the overlay. The strain readings indicate the degree of bonding of the overlay to the existing HMA pavement.

TABLE B1
MN/ROAD WHITETOPPING DESIGN

Section ID	Thickness [mm (in.)]	Joint Spacing [m (ft)]	Fiber Reinforcement
Cell 93	75 (3)	1.2 × 1.2 (4 × 4)	Polypropylene 1.8 kg/m^3 (3 lb/yd^3)
Cell 94	100 (4)	1.2 × 1.2 (4 × 4)	Polypropylene 1.8 kg/m^3 (3 lb/yd^3)
Cell 95	75 (3)	1.5 × 1.8 (5 × 6)	Polyolefin 14.8 kg/m^3 (25 lb/yd^3)
Cell 96	150 (6)	1.5 × 1.8 (5 × 6)	Polypropylene 1.8 kg/m^3 (3 lb/yd^3)
Cell 97	150 (6)	3.0 × 3.6 (10 × 12)	Polypropylene 1.8 kg/m^3 (3 lb/yd^3)
Cell 97b	150 (6)	3.0 × 3.6 (10 × 12) w/dowels	Polypropylene 1.8 kg/m^3 (3 lb/yd^3)

Measurements from the first year show that strains at the bottom of the 75 mm (3 in.) overlay are less than 10 microstrains in tension. Therefore, the overlay is predominately in compression. In the 100 mm (4 in.) overlay, the strains at the bottom are less than 20 microstrains in tension. Flexural rigidity of the system increases as PCC thickness increases, as the neutral axis shifts up, and as more load is carried by the overlay than by the HMA pavement.

Pavement strains are highly dependent on temperature. As temperature decreases, the HMA's resilient modulus increases, and the tensile stresses at the bottom of the overlay increase. Conversely, as temperature increases, the resilient modulus decreases, and the bottom of the PCC overlay can go into compression.

Conclusions

The hardened properties of the fiber-reinforced concrete (FRC) using polyolefin and polypropylene fibers were found to be similar to those of conventional PCC. The bond between the overlay and the underlying HMA is good at the Mn/ROAD site, but subsequent loading will determine if the bond deteriorates. The factors that significantly affect the strains at the bottom of the overlay are HMA stiffness and overlay thickness. The neutral axis of the pavement system can move through the section dependent on temperature changes and on overlay thickness. Increasing the overlay thickness does not always lower the tensile strains at the bottom of the overlay. Instead, they can increase, so it is necessary to select the optimal design from continued testing of the Mn/ROAD sections.

Evaluation of the Mn/ROAD sections has revealed that some of the cracking in the underlying HMA reflected through the UTW layer. This phenomenon is believed to be a function of a high bond strength and stiff HMA layer. It was hypothesized that when the flexural stiffness of the HMA approaches that of the overlay, this distress can occur (*50*).

COLORADO

Whitetopping sections in Colorado, 125 to 175 mm (5 to 7 in.) thick, with joint spacing of up to 3.6 m (12 ft) were instrumented to measure critical stresses and strains owing to traffic loads and temperature differentials (*17,20*). These results were used to develop a thickness design procedure for the state of Colorado's whitetopping overlays (see chapter four of this synthesis for more information). Eleven slabs were instrumented at three different sites in Colorado to obtain the field data. Strain gauges were placed primarily in the center of the slab and along the longitudinal joint. Traffic loads were simulated using Colorado DOT (CDOT) trucks.

Objectives

The objectives for the field testing in Colorado were to

1. Determine the critical stresses and strains in the whitetopping overlays as a result of traffic loads,
2. Examine and measure the interfacial bond between the PCC and the HMA layers, and
3. Experimentally calibrate the theoretical finite-element method (FEM) stress prediction equations.

Experimental Whitetopping Test Sections

Three different Colorado sites (CDOT1, CDOT2, and CDOT3) were instrumented in this study. The PCC mix design for the Colorado whitetopping is given in Table B2 (*19*). Information on whitetopping thickness, HMA thickness, joint spacing, dowels, HMA surface preparation and the modulus of subgrade reaction are given in Table B3. All sections included tie bars along the longitudinal joints.

Field Instrumentation

Field instrumentation varied from site to site. For the first site (CDOT1), all of the test slabs were instrumented with 12 strain gauges. To instrument the interface of the PCC and the HMA, two strain gauges were placed along the transverse centerline on top of the HMA and 12.5 mm (0.5 in.) above the HMA surface in the PCC. These gauges were placed at both longitudinal edges and at the slab center. Gauges were also installed on top of the PCC surface—three above the sets of embedment gauges and three along the corner diagonal line.

TABLE B2
PCC MIX DESIGN FOR A COLORADO WHITETOPPING SECTION

Material	Quantity	
	kg/m^3	lb/yd^3
Type I Cement	335	565
Fly Ash	67	113
Coarse Aggregate	421	710
Fine Aggregate (Sand)	771	1,300
Intermediate Aggregate	546	920
Water	166	280
Air Entrainer (AEA)	913 ml/100 kg cement	14 oz/cwt
Water Reducer	2,217 ml/100 kg cement	34 oz/cwt

AEA = air entraining agent; cwt = hundred weight.

For the slabs at CDOT2, eight strain gauges were used per test slab. Similar to the instrumentation pattern used for CDOT1, the interface between the PCC and the HMA was instrumented at the slab center and at one of the tied longitudinal edges. Likewise, strain gauges were installed above these embedded gauges on the pavement surface.

For the slabs at CDOT3, only four surface strain gauges were used. Two were placed at the slab center and along the longitudinal tied slab edge, and the other two were placed at the slab corner—one in the transverse and the other in the longitudinal direction.

Thermocouples were installed in all the pavements to measure the temperature profile. Five thermocouples were installed through the overlay depth—one at the PCC surface, one at PCC mid-depth, one at the PCC–HMA interface, one 63 mm (2.5 in.) into the HMA, and the last near the bottom of the HMA layer. To measure slab curling, invar reference rods were driven into the subgrade before construction. Relative elevations of the test slabs were recorded using these rods. A Dipstick profiler recorded the test slab profiles.

Laboratory Testing

Cores were taken from the test slabs for determination of layer thicknesses. In addition, the strength of the interfacial bond was measured using the direct shear test. Properties (modulus of elasticity, compressive strength, and flexural strength) of the PCC were measured using the cylinders and beams that were cast during overlay construction.

PCC–HMA Interfacial Bond

The effect of HMA surface preparation on interfacial bond was also examined on sections CDOT1 and CDOT2. This comparison could not be made for the CDOT3 sections because all sections were milled. The results, shown in Table B4, indicate that the interfacial shear strength between the PCC and the milled HMA layer is low at 28 days. The bond between the PCC and the new HMA pavement surface is higher. However, after 1 year, the interfacial shear strengths were found to be comparable.

The effect of milling on new and existing HMA layers was examined using the load-induced strain measurements. For an existing HMA pavement, milling its surface reduces the strain in the whitetopping by 25%. For new HMA pavement, milling increases the strain in the whitetopping by 50%. More research will have to be conducted to examine the use of milling and its effect on the whitetopping interfacial bond and load-induced strains.

Strains at the top of the HMA layer were also measured and compared with the strains at the bottom of the PCC slab. Typically, strains in the HMA layer were less than the strains in the PCC. This finding may indicate that the two layers are not fully bonded.

Stress Results

The location of maximum stress was found to be at the free longitudinal edge (such as a curb and gutter). Because it is unlikely that this joint will experience heavy traffic loading, the tied longitudinal joint was taken as the one with maximum stress.

Profile Results

The profiles of all the instrumented CDOT test slabs were obtained at 28 days and 1 year. Profiles of the 100-mm (4-in.) thick CDOT1 whitetopping sections were obtained along the slab diagonal. It was found that the slab is typically curled upward. The difference in deflection from the slab edge to the slab center is approximately 2.5 mm (0.1 in.) for both the measurements taken at 28 days and 1 year. For the 125-mm (5-in.) CDOT1 whitetopping sections, curling is not as evident and these slabs are relatively flat. Profiles of the CDOT2 slabs are relatively flat as well. For the CDOT3 sections, the difference in displacement between the slab edge and center, as measured on the centerline, is less than 0.5 mm (0.02 in.). The shorter joint spacing is believed to reduce slab curling.

Pavement Loading

The pavement was loaded several times during the day to measure the load-induced stresses as a function of the temperature differential. This correlation was made so that the load-induced stress at a zero-temperature gradient could be obtained.

TABLE B3
THREE COLORADO FIELD TEST SITES FOR WHITETOPPING CONSTRUCTION

Name	Slab No.	Whitetopping Thickness [mm (in.)]	HMA Thickness [mm (in.)]	Joint Spacing [m (in.)]	Dowels	HMA Surface Preparation	Modulus of Subgrade Reaction (psi/in.)
CDOT1	1	119 (4.7)	114 (4.5)	1.47 × 1.47 (58.0 × 58.0)	N	New	150
	2	147 (5.8)	150 (5.9)	1.46 × 1.46 (57.6 × 57.6)	N	New	150
	3	152 (6.0)	137 (5.4)	1.49 × 1.49 (58.5 × 58.5)	N	New milled	150
CDOT2	1	130 (5.1)	84 (3.3)	1.85 × 1.85 (73.0 × 73.0)	N	Existing	340
	2	137 (5.4)	117 (4.6)	3.80 × 3.07 (149.5 × 121.0)	N	New	340
	3	160 (6.3)	86 (3.4)	1.89 × 1.82 (74.5 × 71.5)	N	New	340
	4	185 (7.3)	86 (3.4)	3.80 × 1.83 (149.8 × 72.0)	N	Existing milled	340
	5	173 (6.8)	71 (2.8)	3.80 × 3.92 (149.8 × 154.5)	Y	Existing milled	340
CDOT3	B	188 (7.4)	178 (7.0)	3.05 × 3.58 (120.0 × 141.0)	Y	Existing milled	225
	E	173 (6.8)	168 (6.6)	1.80 × 1.83 (71.0 × 72.0)	Y	Existing milled	225
	F	142 (5.6)	168 (6.6)	1.82 × 1.75 (71.5 × 69.0)	Y	Existing milled	225

Notes: N = no; Y = yes.

Findings

A number of interesting and important observations were made at the Colorado experiments. For TWT overlays, it was found that tie bars were helpful in maintaining horizontal and vertical alignment between adjacent lanes along the construction joint.

TABLE B4
EFFECT OF HMA SURFACE PREPARATION ON PCC–HMA INTERFACIAL BOND

Name	Slab No.	HMA Surface Preparation	28-Day Interfacial Shear Strength [kPa (psi)]	1-Year Interfacial Shear Strength [kPa (psi)]
CDOT1	1	New	310 (45)	552 (80)
	3	New milled	69 (10)	552 (80)
CDOT2	1	Existing	689 (100)	—
	4	Existing milled	448 (65)	689 (100)
	5	Existing milled	—	1,069 (155)

In addition, it was found that, at least initially, the bond strength between the PCC and the HMA is very low if a new HMA level up course is used instead of placing it directly atop the existing surface. The additional observation of a "tender" HMA mix should also be considered. This finding was reversed as the newer HMA aged and the bond was later tested. The bond strength appeared to increase over time, and it was found to be equal to or better than the bond of the PCC to existing HMA.

IOWA

The state of Iowa, through the Iowa DOT, has provided valuable information for the understanding of variables affecting the performance of whitetopping overlays. Whitetopping experimental projects conducted by Iowa include one conducted on Road R16 to study the effect of different preparation methods to enhance the bond strength between the existing HMA pavement and the PCC overlay. A second effort, on Route 21, investigated the effect on performance by a number of variables.

Road R16 Project

A whitetopping test section was constructed on Road R16, in Dallas County, to investigate the effectiveness of various surface preparation techniques in improving the bond strength between the existing HMA pavement and the PCC overlay. The existing pavement, built in 1959, consisted of a 63-mm (2.5-in.) HMA surface on top of a 150-mm (6-in.) rolled stone base, over 100 mm (4 in.) of soil base. In 1971, a 75-mm (3-in.) HMA overlay was constructed. The initial condition of the pavement surface included ruts in excess of 25 mm (1 in.), random and transverse cracking, and some areas with alligator cracking. Twelve test sections were constructed, as shown in Table B5 (*79*).

Data Collected

Data collected on this research effort include rut-depth measurements, Road Rater structural measurements, beam and cylinder strengths, core shear strengths, slump, and entrained air. Compressive strengths in the range of 23.2 MPa (3,364 psi) to 28.4 MPa (4,118 psi) and flexural strengths in the range of 4.34 MPa (629 psi) to 4.75 MPa (689 psi) for Sections 2 to 5 were reported. The average shear strengths obtained ranged from 600 kPa (87 psi) to 1,500 kPa (218 psi). Some of the cores extracted presented no bond at the PCC–HMA interface, or the HMA was broken into pieces.

In the years of 1994 and 1996, additional cores were extracted to test for shear strength at the PCC–HMA interface. Mack et al. (*77*) contains a detailed table with shear strength information, location, and pavement thicknesses at the core locations for a total of 142 cores. Pavement condition surveys were performed in 1992, 1994, and 1996.

Findings

From results shown in Table B6, it can be concluded that sections with milling as surface preparation developed higher bond shear strengths. In addition, it was observed that tack coat might reduce strength when a cationic emulsion is used. Sections with cement and water grout as a bonding agent did not show any contribution to bond strength. Different PCC types, thicknesses, planing, and air blasting did not affect bond strength significantly.

It was also found in the study that the increase in shear strength does not correlate to an increase in structural contribution of the old HMA pavement. According to the study, only sufficient bond is necessary to anchor the PCC to the old HMA, to use some of the underlying HMA pavement structure. It was concluded that the structural evaluation did not provide enough information to determine the bond strength or the level of support provided by the HMA layer.

In subsequent monitoring of the test sections, it was found that the PCC–HMA bond was degrading over time in the outside wheelpath for all bonding methods, except tack coat. The total pavement thicknesses correlated very well with the pavement structural capacity tested with the Road Rater. The primary observations from the condition surveys are that

TABLE B5
EXPERIMENTAL DESIGN FOR THE IOWA ROAD R16 PROJECT

Section*	Surface Preparation	Bonding Agent	Planing	Design Thickness (mm)	As-Built Thickness (mm)	Mix (Iowa DOT classification)
2	Broomed	None	No	130	124	B
3	Milled	None	No	100	106	B
4	Milled	None	No	100	117	C
5	Milled	None	Yes	100	125	C
6	Broomed	None	No	100	114	C
7	Broomed	None	No	130	142	C
8	Broomed w/air blast	None	No	130	143	C
9	Milled	None	No	130	149	C
10	Milled	None	Yes	130	153	C
11	Milled	Cement and water grout	No	130	146	C
12	Broomed	Cement and water grout	No	130	135	C
13	Broomed	Tack emulsion	No	130	133	B

*Section 1 is not mentioned in the reference (*79*).

TABLE B6
AVERAGE SHEAR STRENGTH AND SURFACE PREPARATION

Section	Core Shear Strength in 1991 [MPa (psi)]	Surface Preparation	Contribution of Old HMA (SN)
5	10.3 (1,500)	Milled	1.08
3	9.0 (1,300)	Milled	1.15
8	8.3 (1,200)	Broomed w/air blast	1.35
4	7.9 (1,150)	Milled	1.14
11	6.9 (1,000)	Milled	1.26
2	6.6 (950)	Broomed	1.38
9	6.6 (950)	Milled	1.51
12	6.2 (900)	Broomed	1.46
10	6.2 (900)	Milled	1.79
6	5.5 (800)	Broomed	1.54
7	5.5 (800)	Broomed	1.25
13	4.1 (600)	Broomed	0.97

SN = structural number. *Source*: Grove et al. (*79*).

- Most cracks observed were longitudinal, attributed to weakness of the base; and
- The cracks are concentrated in Sections 11, 6, 3, and 4 (in decreasing order).

No connection was found between the cracking and the surface preparation methods used. However, some correlation was found between cracking and PCC thickness and between cracking and planing. It was found that cracking was predominant on the thinnest PCC sections that were not planed.

Route 21 Project

In 1994, an experimental whitetopping overlay was constructed in Iowa County. The project, 11.6 km (7.2 mi) in length, was constructed on Iowa Route 21, which carried average daily traffic of approximately 1,000 at the time. The objective of the project was to evaluate the performance of various whitetopping sections with a variety of design factors, including PCC thickness, use of fibers, and joint spacing.

The project consists of 41 test sections with PCC thicknesses ranging from 50 to 200 mm (2 to 8 in.) overlaying an 88-mm (3.5-in.) HMA surface, 175-mm (7-in.) cement-treated base, and 150 mm (6 in.) of granular subbase (rolled stone base) built in 1961. In addition, 24 sections were used to transition between test sections. Different slab lengths were used, from 0.6 to 4.5 m (2 to 15 ft). Sections with PCC thicknesses greater than 100 mm (4 in.) had their joints sealed with hot-pour sealant. Sections with thicknesses of 100 mm (4 in.) or less were not sealed, except for five sections at a rate of 1.8 kg/m^3 (3 lb/yd^3). Monofilament and fibrillated polypropylene fibers were added to the PCC mix for designated sections. Three test sections were overlaid with 113 mm (4.5 in.) of HMA for comparison purposes. The typical section was 213 m (700 ft) in length. Soil type for the length of the project, classified according to both the Unified Classification System and AASHTO, is detailed elsewhere (*114*).

Conditioning of the pavement for the whitetopping consisted of patching and scarifying, patching only, and cold in-place recycling (CIPR). For the CIPR, 95 mm (3.75 in.) was removed and combined with 2.3% CSS-1 emulsion (by weight of material). This rejuvenated material was placed back onto the milled surface 1 month in advance of the construction of the whitetopping sections. The type of fibers used in each section is presented in Table B7, and the experimental design for this project is summarized in Table B8.

Although it was not initially considered, the contractor sprayed the HMA surface with water immediately ahead of the paver. This procedure was stopped after the first 15 sections had been constructed. The difference in the first 15 sections is therefore considered as a variable inadvertently introduced. As of 1994, the average annual daily traffic was 1,090 vehicles per day (vpd) and average annual truck traffic was 142 vpd.

Data Collected

The information obtained during placement of the whitetopping project is presented in Wilde et al. (*116*) and includes the following:

- Daily inspection reports of PCC;
- Daily plant reports for HMA;
- Sawcut time;
- Slump and air;
- Beam and cylinder strengths at 7, 14, and 28 days;
- Profilograph results;
- Slab thicknesses;
- Paver vibrator revolutions per minute;
- Concrete and air temperatures at the time of placement;
- Documentation of distresses made with photographs and condition surveys;

TABLE B7
TYPE OF FIBERS USED FOR EACH SECTION

Section	From	To	Type
1–2	2335+6	2341+0	Convention
2–10	2341+0	2386+7	Fibrillated
10–14	2386+7	2412+7	Monofilam
14–15	2412+7	2415+0	Fibrillated
17–33	2425+0	2505+0	Convention
35–37	2515+0	2539+0	Convention
37–54	2539+0	2632+2	Fibrillated
54–64	2632+2	2703+9	Convention

Source: Heyer and Marks (*119*).

TABLE B8
SUMMARY OF EXPERIMENTAL SECTIONS FOR IOWA ROUTE 21 PROJECT

Section Number	Begin Station	End Station	Length (ft)	Design Thickness (in.)	As-Built Thickness (in.)	Fiber	Joint Spacing (ft)	Surface Prep.
1	2335+60	2340+00	440	8	200	N/A	20	N/A
3	2342+00	2349+00	700	6	150	F	12	P&S
4	2349+00	2356+00	700	6	150	F	6	P&S
6	2357+00	2364+00	700	4	100	F	6	P&S
7	2364+00	2371+00	700	4	100	F	2	P&S
8	2371+00	2378+00	700	4	100	F	4	P&S
10	2380+00	2387+00	700	2	50	F	2	P&S
11	2387+00	2394+00	700	2	50	F	4	P&S
13	2396+00	2403+00	700	6	150	F	6	P&S
14	2403+00	2414+00	1100	6	150	F	12	P&S
16	2415+00	2425+00	1000	4.5	110	HMA	N/A	P&S
18	2426+00	2433+00	700	6	150	NF	12	P&S
19	2433+00	2440+00	700	6	150	NF	6	P&S
21	2441+00	2448+00	700	4	100	NF	2	P&S
23	2449+00	2456+00	700	2	50	NF	2	P&S
25	2458+00	2460+00	200	6	150	NF	6	P&S
26	2460+00	2468+00	800	6	150	NF	6	P Only
27	2468+00	2479+00	1100	6	150	NF	12	P Only
29	2480+00	2487+00	700	4	100	NF	4	P Only
31	2489+00	2496+00	700	8	200	NF	15 ND	P Only
32	2496+00	2503+00	700	8	200	NF	15 D	P Only
34	2505+00	2515+00	1000	4.5	110	HMA	N/A	P Only
36	2516+00	2538+00	2200	6	150	NF	6	P Only
38	2540+00	2547+00	700	2	50	F	2	P Only
39	2547+00	2554+00	700	2	50	F	4	P Only
41	2555+00	2562+00	700	4	100	F	4	P Only
42	2562+00	2569+00	700	4	100	F	2	P Only
43	2569+00	2576+00	700	4	100	F	6	P Only
45	2577+00	2585+00	800	6	150	F	12	P Only
46	2585+00	2593+00	800	6	150	F	6	CIPR
48	2594+00	2601+00	700	4	100	F	6	CIPR
49	2601+00	2608+00	700	4	100	F	2	CIPR
50	2608+00	2615+00	700	4	100	F	4	CIPR
52	2616+00	2624+00	800	2	50	F	2	CIPR
53	2624+00	2631+00	700	2	50	F	4	CIPR
55	2633+00	2640+00	700	6	150	NF	6	CIPR
56	2640+00	2653+00	1300	6	150	NF	12	CIPR
58	2654+00	2661+00	700	4	100	NF	6	CIPR
60	2662+00	2689+00	2700	6	150	NF	12	CIPR
62	2691+00	2698+00	700	2	50	NF	4	CIPR
65	2704+00	2714+08	1008	4.5	110	HMA	N/A	CIPR

Notes: N/A = not applicable; P&S = patch and scarify (milling); P Only = patch only; CIPR = cold-in-place recycle; D = dowels; ND = no dowels; F = fibers present; NF = no fibers present.
Source: Heyer and Marks (*119*).

- Pullout (pull-off) testing; and
- Structural ratings with a road tester.

Traffic loading was monitored with a weigh-in-motion device installed in each lane.

Condition Surveys

Late sawed joints resulted in transverse cracking at Sections 36, 39, 41, 43, and 45. The rest of the overlay did not show transverse cracks caused by late sawcutting. No change was observed from initial construction to approximately 1.5 years after construction. During the second year, new cracks were observed that have been attributed to severe temperature changes. Figures B1 through B5 present the distress progression during the first 2 years. Only sections that had cracking during that period are plotted.

Potential debonding of the test sections was monitored with manual soundings. Only Sections 23 and 62, with a PCC thickness of 50 mm (2 in.), showed possible debonding deterioration. In summary, the following observations were made for the condition surveys during the first 2 years:

- Only sections with 50 and 100 mm (2 and 4 in.) of PCC indicated some distress.
- Only two of the eight sections with 50 mm (2 in.) PCC thickness exhibited some debonding and cracking

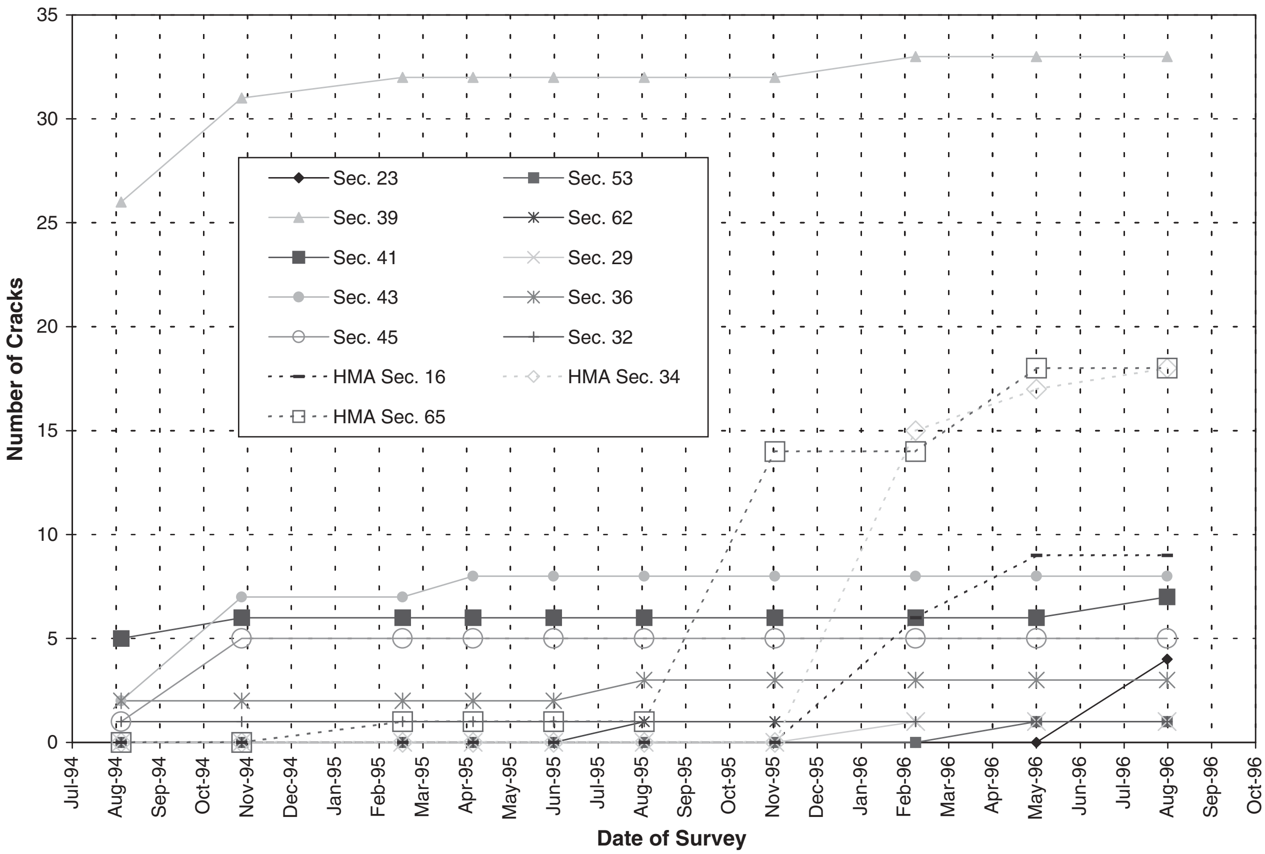


FIGURE B1 Iowa Route 21—transverse cracking.

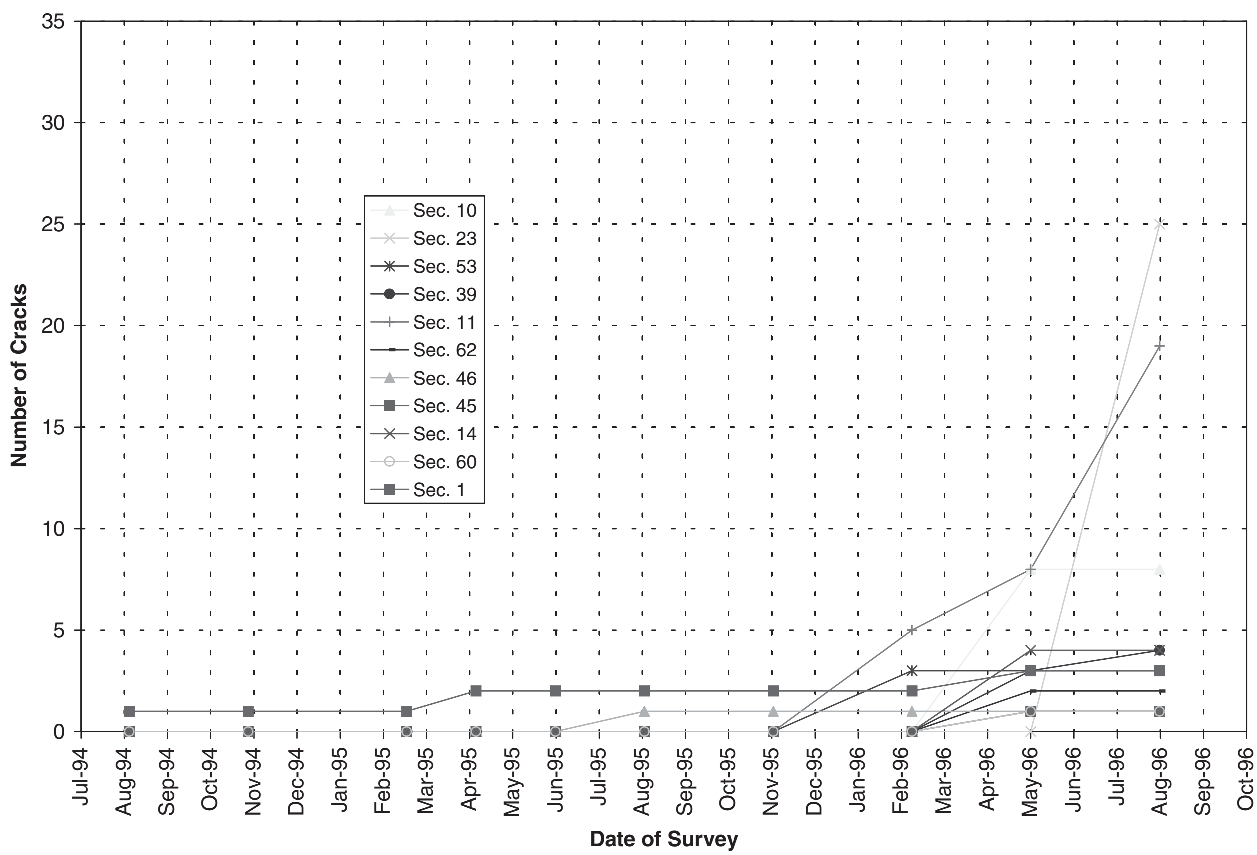


FIGURE B2 Iowa Route 21—longitudinal cracking.

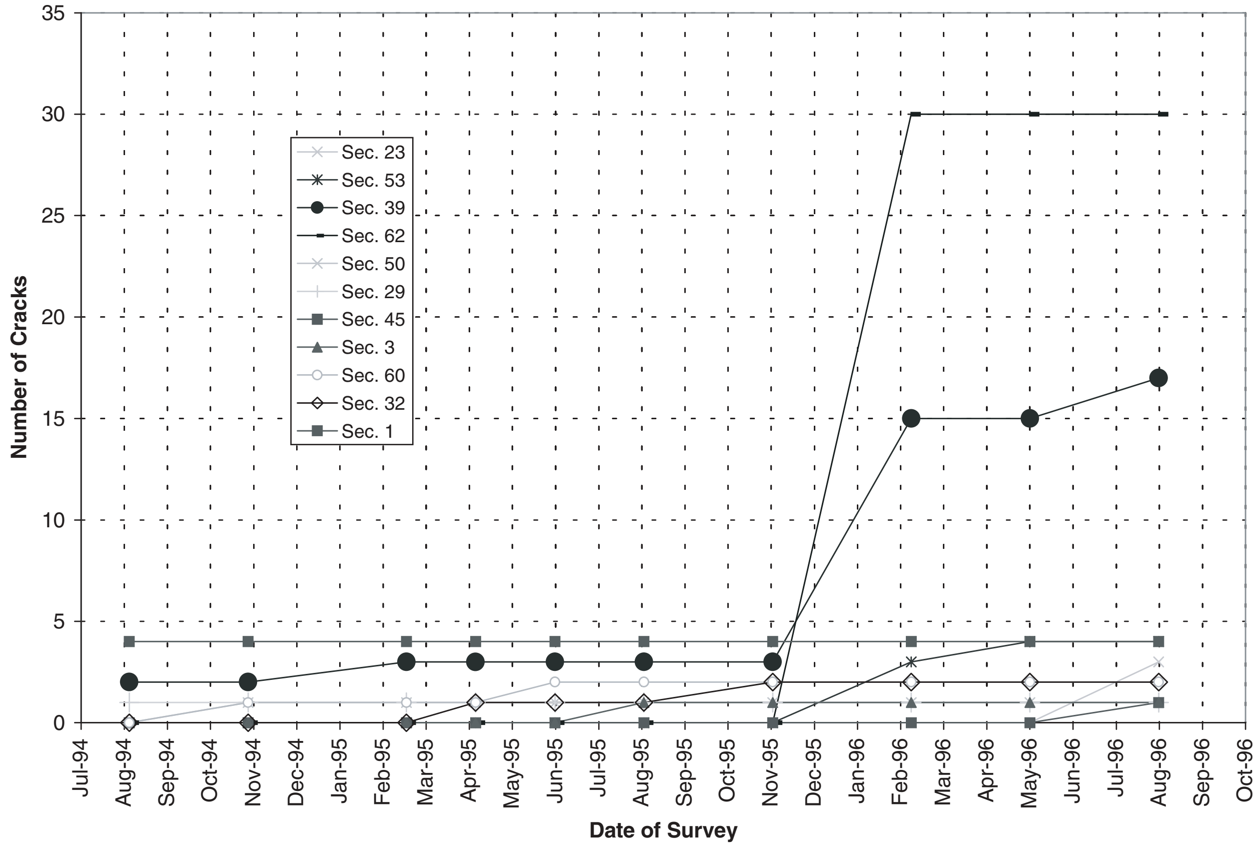


FIGURE B3 Iowa Route 21—corner cracking.

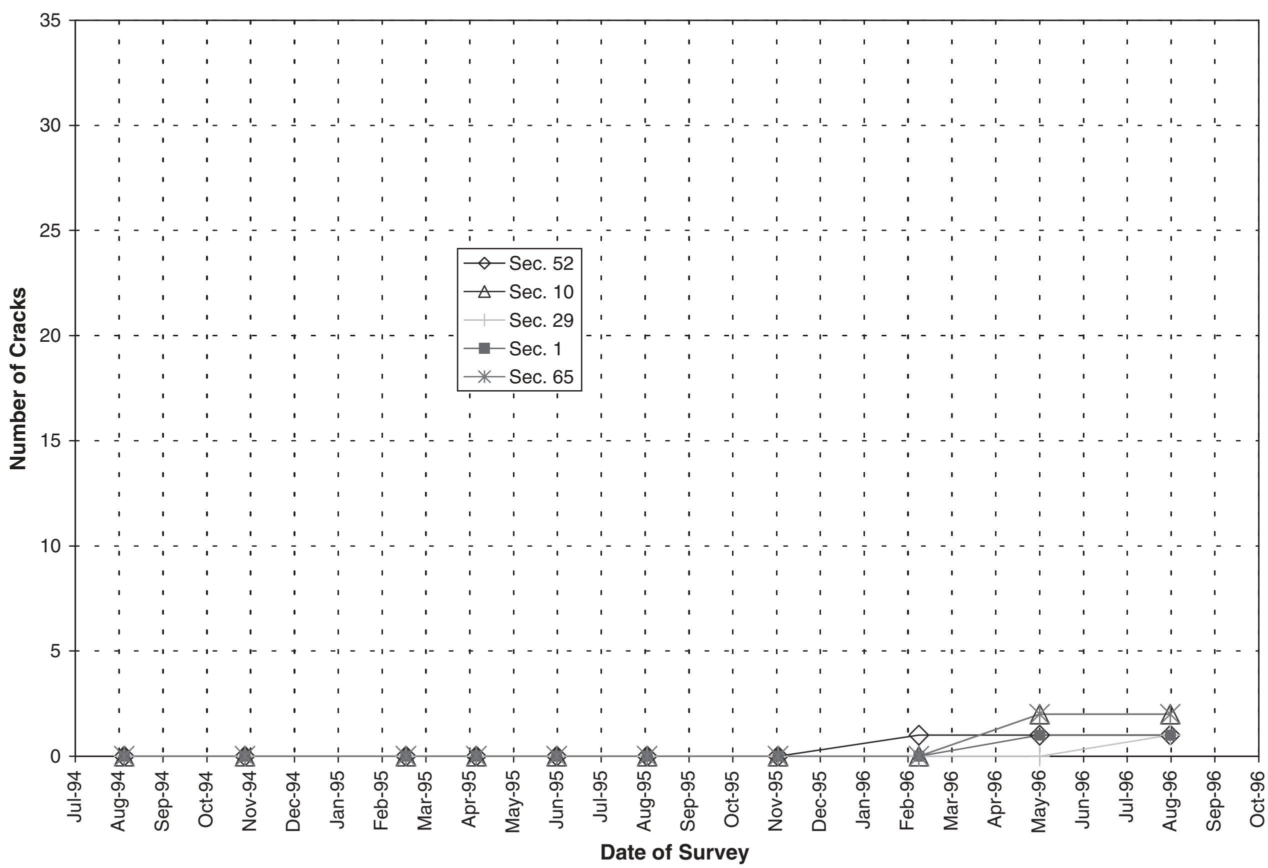


FIGURE B4 Iowa Route 21—diagonal cracking.

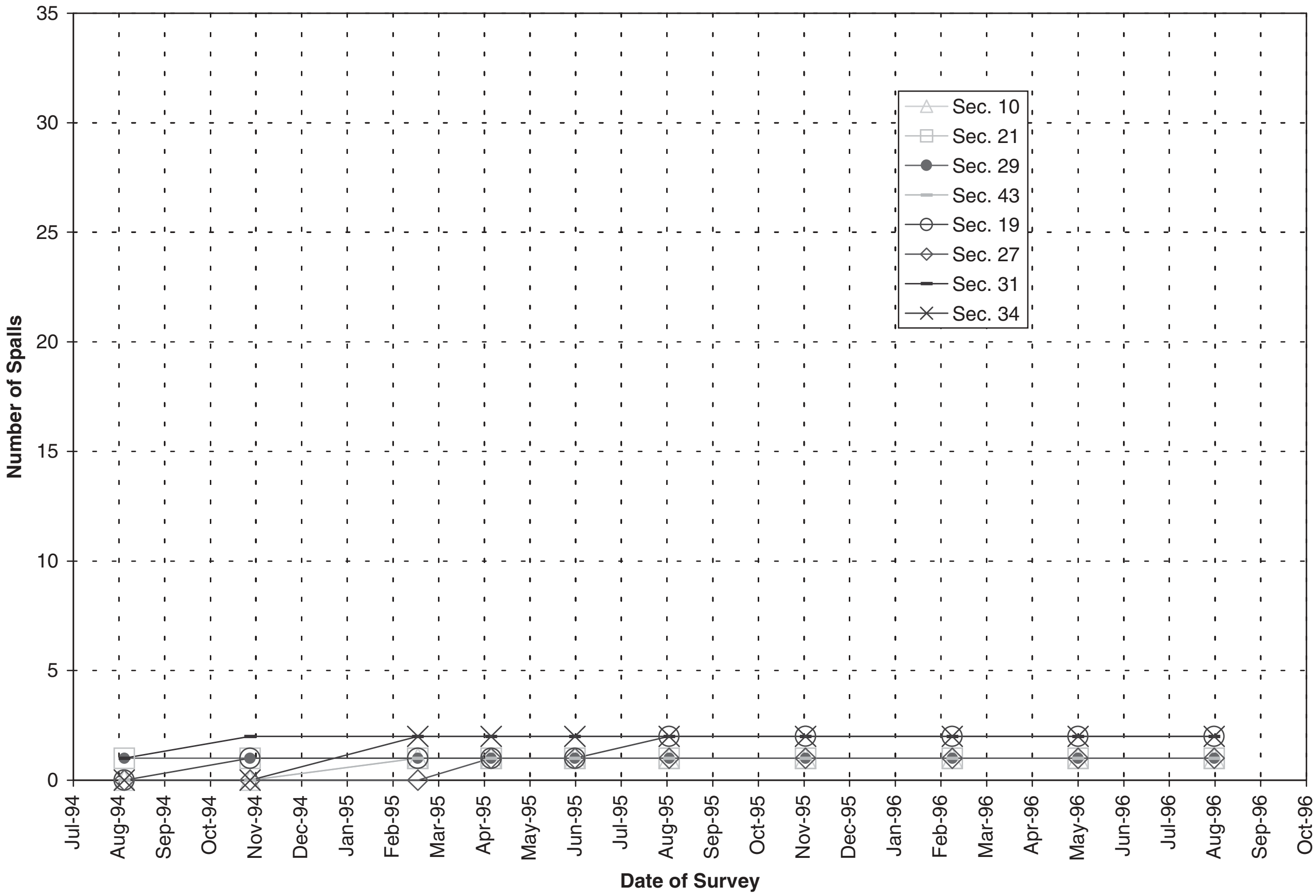


FIGURE B5 Iowa Route 21—spalling.

distress. The area that presented distress cracking is only 2% for each section.

- The two 50 mm (2 in.) sections with debonding problems and cracking distress contained no fibers. Two 50-mm (2-in.) thick sections with cracking, but no debonding, contained fibers. The remaining four 50-mm (2-in.) thick sections with no cracking or debonding contained fibers.
- One of the sections with debonding and cracking was reported to have a construction problem.
- Transverse cracking has been observed in the three HMA sections. Cracking at those sections started approximately 1.5 years after construction.

Laboratory Testing

Sixty-four PCC–HMA composite beams were fabricated and tested under static and flexural loading. For this experiment, a factorial was used to test for different conditions, including HMA surface preparation (milled or not milled), PCC thickness of 50 or 100 mm (2 or 4 in.), and use of fibers.

Instrumentation

The instrumentation for this project included the use of deflectometers assembled to monitor PCC strains. The instrumentation was performed before paving operations at 35 of the 41 PCC sections. Sections 1, 16, 25, 34, 45, and 65 were not instrumented. No instrumentation was performed on the HMA sections. At each site, two deflectometers and a thermocouple were installed.

Deflection Testing

Deflection testing was performed with falling weight deflectometer (FWD) equipment before and after construction of the whitetopping section. Before construction, the existing pavement was tested on the outer wheelpath at 91 m (300 ft) intervals and at the instrumentation locations. After construction, deflection testing was performed at the center of the panels located in the outer wheelpath of the instrumented lane.

Findings

After 2 years of performance, the following conclusions were made (*48*):

- All whitetopping sections have performed well.

- Initial distresses on the 50 mm (2 in.) and 100 mm (4 in.) sections may indicate that 50 mm (2 in.) sections are more affected owing to construction procedures or weaknesses in the base.
- Milling seems to enhance bonding and reduce distress for all pavement sections.
- The thinnest sections with fibers have showed better performance than did similar sections with no fibers.

No results on the laboratory testing of the composite beams, monitoring of the instrumented sections, or results from the deflection testing, were found in the reports reviewed for this project.

TENNESSEE AND GEORGIA

A number of UTW sections have been constructed in the states of Tennessee and Georgia. The performance of these sections has been observed at regular intervals.

Nine UTW projects, built from 1992 to 1994, were selected for pavement condition monitoring at frequent intervals (*10*). Data on these sections are presented in Table B9. Differences in joint design and performance resulted in the division of Cusick Street into two different sections: Cusick Street (outside lane) and Cusick Street (inside lane). A similar division for the I-85 weigh station was also warranted owing to differences in mix design into "approach side" and "leave side."

TABLE B9
TENNESEE AND GEORGIA UTW SECTIONS

Location	City/County	State	When Built	Joint Spacing [m (ft)]	Thickness [mm (in.)]	Fibers	Observations as of September 2000
Belvoir Ave. and Brainerd Rd.	Chattanooga	TN	Nov. 93	1.5 (5)	64–75 (2.5–3)	Yes	Still functioning
28th Street	Chattanooga	TN	July 92	1.5 × 1.1 (5 × 3.75)*	64–75 (2.5–3)	Yes	Most HMA layer was removed during milling. Overlaid with ACC 4 years later
Green St. and North Jackson St.	Athens	TN	Jan. 94	0.9 × 1.1 (3 × 3.67)*	64 (2.5)	Yes	Still performing very well
SH 56	McMinnville	TN	Sep. 93	0.9 (3)	64–75 (2.5–3)	Yes	No observations
Concorde St. and Kingston Pk.	Knoxville	TN	Nov. 92	1.2 × 1.0 (4 × 3.3)*	89–100 (3.5–4)	Yes	Still in very good condition
Cusick St. and Harper Ave.	Maryville	TN	Aug. 93	1.2 × 1.5 (4 × 5)*	65–75 (2.5–3)	Yes	No observations
I-85 Weigh Station	Near Lavonia	GA	May 93	0.6 (2)	50 (2)	F & NF	No observations
Wesley Chapel Rd.	DeKalb County	GA	Sep. 93	1.2 (4)	64 (2.5)	Yes	No observations
Marbut Rd. and Lithonia I. Blvd.	DeKalb County	GA	Sep. 93	1.2 × 1.1 (3.83 × 3.5)*	64–75 (2.5–3)	Yes	No observations

*Transverse—Longitudinal joint spacing.
Sources: Cole (*10*) and J. Norris, personal communication (Information on UTW Projects in Tennessee) to T. Ferragut, Sep. 4, 2000.

Construction Procedures

Typical construction procedures reported include the following (*10*):

- All the UTW projects had similar mixture proportions. Typical Tennessee mixture proportions are reported in Table B10.
- The existing HMA surface was milled and broomed.
- No bonding agent was applied.
- Ready-mixed concrete was used and placed with a vibrating screed.
- The surface was bull floated and textured with a broom.
- White-pigmented curing compound was applied and joints were sawed with early sawing techniques.
- Ambient temperature varied from project to project from less than 4.4°C (40°F) to more than 32.2°C (90°F) during placement. Sherwood (*141*) provides additional detail on placement temperatures.

Traffic Loading

Of all the UTW sections, only I-85 has detailed traffic information available. The traffic counts for the other sections were based on several traffic counts (Green Street, Tennessee SH 56, Concorde Street, and Cusick Street) or estimated from observations (Belvoir Avenue, Wesley Chapel Road, and Marbut Road). Table B11 presents the estimated daily traffic and ESALs for the years of 1995 and 1996 for all the UTW sections, except 28th Street.

Data Collected

All UTW sections were surveyed using the PAVER System protocol. Cracking condition surveys and pavement condition index (PCI) using the PAVER System protocol were taken for each year from 1995 through 1998. The PAVER System protocol uses a numerical index from 0 (Very Poor) to 100 (Excellent) to rate the condition of the pavement. A summary of the performance of the UTW sections in terms of cracking and the PCI is given in Table B12. A detailed summary of the different types of cracking per section for the years 1995 and 1996 is presented by Cole (*10*).

TABLE B10
TYPICAL MIX PROPORTIONING FOR TENNESSEE UTW SECTIONS

Constituent	Dosage
Cement	802 lb
Coarse Aggregate #7	1,711 lb
Fine Aggregate	1,098 lb
Water	280 lb
Water Cement Ratio	0.35
Fibers	3 lb
Superplasticizer	15 oz
Air Entrainment	6 oz

Source: J. Norris, personal communication (Information on UTW Projects in Tennessee) to T. Ferragut, Sep. 4, 2000.

TABLE B11
ESTIMATED ACCUMULATED TRAFFIC LOADING

Project	Vehicles (millions)	Trucks (thousands)	ESALs in 1998 (thousands)
Belvoir Ave. and Brainerd Rd.	3.35	134	101
Green St. and North Jackson St.	6.9	414	311
State Highway 56	8.47	508	381
Concorde St. and Kingston Pike	4.40	176	132
Cusick St. and Harper Ave. (outside)	10.05	402	302
Cusick St. and Harper Ave. (inside)	6.98	268	201
I-85 Weigh Station (approach)	0.16	160	650
I-85 Weigh Station (leave)	0.16	160	650
Wesley Chapel Road	9.26	463	347
Marbut Road and Lithonia I. Blvd.	1.22	122	92

Note: From construction date to condition survey date. *Source*: Cole (*78*).

Findings

The following findings were based on the evaluation of the UTW sections:

- Corner cracking, linear cracking, and divided slabs are the most prominent distress types.
- All UTW sections, except 28th Street in Chattanooga, Tennessee, presented mostly low-severity cracking. The UTW on 28th Street was surveyed after 3 years, showing a poor condition with a PCI of 32. This was attributed to the very poor condition of the existing HMA pavement or absence of this layer according to cores taken in that project. That section was not surveyed in later years.
- Comparison of the PCI with age shows that the PCI is decreasing with time, as would be expected.
- Comparison of the PCI with ESALs shows that PCI is decreasing with increasing ESALs. However, three of the four sections with the lowest PCI also have low ESAL accumulations. The two sections with the highest PCI have the most ESALs accumulated, as is the case for I-85. Good performance of the sections with the most ESALs accumulated may be attributed to the depth of the HMA and the joint spacing, which for I-85 is approximately 275 mm (11 in.) and 0.6 m (2 ft), respectively. A more detailed investigation of other factors affecting the performance of those test sections is required.
- A significant concentration of the cracking observed occurred near the approach or leave ends of the UTW sections. The higher concentration of cracking at the ends of the UTW sections is potentially attributed to

TABLE B12
CONDITION OF UTW TENNESSEE AND GEORGIA SECTIONS

Location	Percentage of Panels with Cracks				Pavement Condition Index			
	1995	1996	1997	1998	1995	1996	1997	1998
Belvoir Ave.	13	21	21	ND	90	87	85	80
Green St.	3	8	8	9	98	95	92	89
State Highway 56	9	15	19	19	89	86	85	83
Concorde St.	2	3	4	5	97	96	96	95
Cruisick St. (outside)	15	16	16	16	93	91	89	89
Cruisick St. (inside)	44	51	58	58	75	71	66	66
I-85 Weigh Station (approach)	2	2	3	4	98	98	98	97
I-85 Weigh Station (leave)	7	7	9	20	95	94	95	74
Wesley Chapel Rd.	7	11	12	ND	96	92	91	ND
Marbut Rd.	4	7	8	10	95	91	89	88

ND = no data. *Source*: Cole (*78*).

impact loading, free edge condition, and debonding of the PCC overlay.
- It was observed that the sections with the highest percentage of cracking also have the largest panel size.

MISSOURI

Several test slabs at the Spirit of St. Louis Airport were instrumented at the time of construction (February 2–3, 1995), so that the behavior of whitetopping overlays could be assessed. This facility is a general aviation airport located in Chesterfield, Missouri. Similar to the experimental work done in Colorado (see the earlier section on Colorado), strain gauges were installed, as were thermocouples for the measurement of critical stresses and strains caused by traffic load and by changing climatic temperatures. One focus of this experiment is measuring the bond strength between the PCC and the HMA.

Field Instrumentation and Testing

Six slabs were instrumented and load tested at the airport. Slab dimensions were 88 × 1250 × 1250 mm (3.5 × 50 × 50 in.), and polypropylene fibers were used as microreinforcement. Mix proportions are given in Table B13.

The joint design was varied in this series of experiments. Three different joints were used. One joint type was a normal contraction joint over an HMA layer with no cracks. Another joint type was designed to simulate the free edge condition (the joint was sawed to the bottom of the HMA layer). The final joint type was designed to simulate isolated test slabs, with all four joints sawed to the bottom of the HMA layer.

Location of Strain Gauges

Plan views of the embedment strain gauge locations are shown in Figure B6. The embedment strain gauges were placed at the slab centers and joints on the surface of the HMA layer and 1 in. (25 mm) above the surface of the HMA in the PCC.

TABLE B13
MIX PROPORTIONS USED IN THE WHITETOPPING SECTIONS AT THE SPIRIT OF ST. LOUIS AIRPORT

Material	Quantity	
	kg/m³	lb/yd³
Type I Cement	303	510
Class C Fly Ash	47	80
Coarse Aggregate (#57 limestone)	1,115	1,879
Fine Aggregate (sand)	750	1,263
Water	110–113	183–190
Air Entrainment	359 ml/100 kg cement	5.5 oz/cwt
Polypropylene Fibers	1.79	3
Low-Range Water Reducer	1108 ml/100 kg cement	17 oz/cwt

Thermocouples were used to measure the temperature profile through the thickness of the whitetopping overlays. Thermocouples were placed at the top, middle, and bottom of the PCC overlay and 12.5 mm (0.5 in.) into the HMA layer.

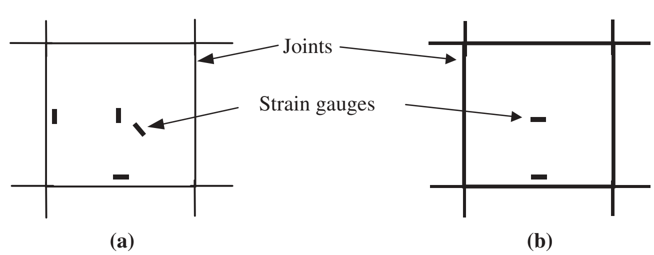


FIGURE B6 Plan view of the location of embedment strain gauges: (a) typical pattern; (b) pattern for slabs with four free edges.

Interfacial Bond Between PCC and HMA

The existing HMA pavement was milled and cleaned by air blasting before PCC placement. Bond strength between the PCC layer and the HMA layer was measured by using both the interface bond strength (direct shear) test and the pull-off test for tensile bond strength. The average value of the interfacial shear strength is 0.70 MPa (102 psi). Past studies have indicated that bond strength of 0.69 MPa (100 psi) is sufficient. The average value of the direct tensile pull-off test was 0.51 MPa (74 psi).

Using the data from the strain gauges, the bond between the PCC and the HMA layer can be assessed. The strain at the bottom of the PCC is extrapolated from the readings obtained at the surface and 25 mm (1.0 in.) from the bottom of the PCC layer. This extrapolated strain is compared with the measured strain at the surface of the HMA layer. The difference between these two strains ranges from 7.9 to 8 microstrain regardless of the joint condition. This finding was interpreted to be a result of the partial bonding of the PCC layer to the underlying HMA layer at the Spirit of St. Louis Airport.

Slab Surface Profile

A Dipstick profiler was used the morning after construction to record relative elevations. A plot of relative elevation versus diagonal distance for the slab with no free edges is shown in Figure B7.

To obtain the degree of curling, a linear regression of the data was first performed to ascertain the slope of the slab (constructed for drainage). Then this linear regression was subtracted from the elevation data to obtain the difference. Because this difference contains components of slab curling, construction roughness, and interaction with adjoining slabs, it is difficult to extract only the curling profile. A quadratic regression was performed and the difference in relative elevations between the diagonal corner and the slab center was 0.89 mm (0.035 in.). A similar analysis was performed on the whitetopping slabs having one free edge and four free edges; the extracted amount of curling was 0.79 mm (0.031 in.) and 0.64 mm (0.025 in.), respectively. This analysis showed that these slabs had a curled-downward profile.

Curling of the slabs was measured again in May and September. It is assumed that the relative movement of the slabs was measured throughout the day by using a Dipstick and the early morning measurements were used as the reference point. The maximum differences in the relative movements ranged from 0.36 to 0.64 mm (0.014 to 0.025 in.) for all the slabs. For a typical pavement with thickness of 250 mm (10 in.), the maximum difference can be 2.5 mm (0.10 in.). Therefore, the whitetopping slabs appear not to be lifting off the HMA layer but are maintaining good contact with the foundation. The short joint spacing contributes to the good contact between the PCC and the HMA layer.

Load Testing

Before load testing, surface gauges are attached to the PCC surface in the pattern, as shown in Figure B8. Gauges are placed on the unloaded side of the slab for determining load transfer efficiency. Loading was provided by a rental truck having a front-axle load of 16 kN (3.6 kips) and a rear-axle load of 44.5 kN (10 kips) for the May test. For the September test, the rear axle load was increased to 53 kN (11.9 kips). Load positions are also shown in Figure B8.

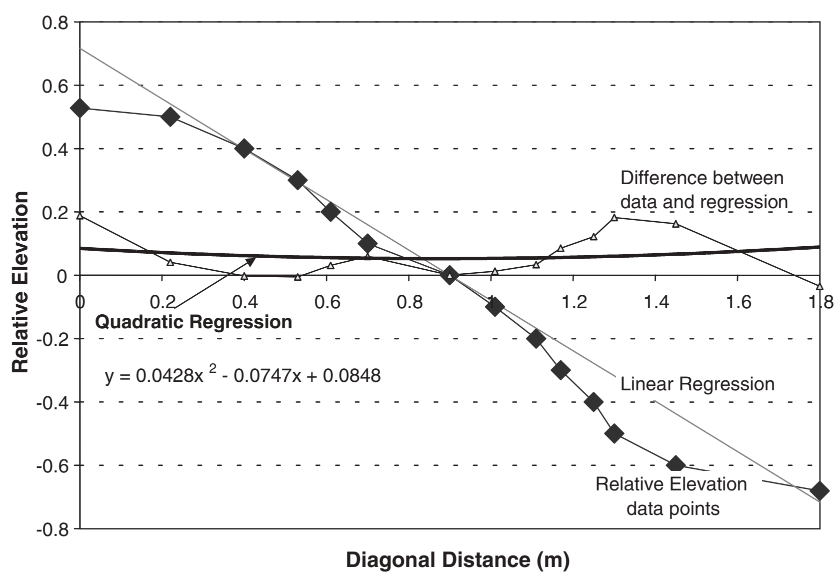


FIGURE B7 Plot of relative elevation versus diagonal distance for whitetopping slab with no free edges.

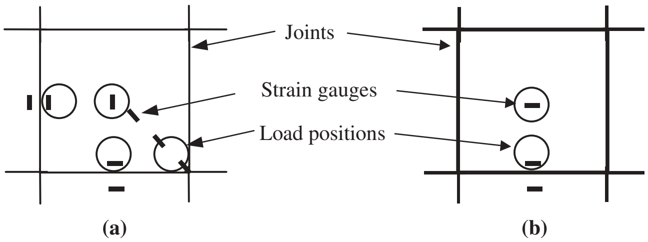


FIGURE B8 Pattern for surface strain gauges and truck loading: (a) typical pattern; (b) pattern for slabs with four free edges.

The strains in all the gauges for the load testing in May and September are reported by Wu et al. (*19*). If none of the joints is cut, it was found that the stresses in the middle of the slab are comparable to the ones at the slab edges. It was also found that strain was higher in the pavements with cut joints than in those without free joints.

Load transfer at the joint in the whitetopping slabs was also examined. Load transfer was calculated based on stress. The stress on the unloaded side of the slab was divided by the stress on the loaded side of the slab. In May, load transfer was found to be 69% for the normal joints. For the slab with four free joints load transfer was 41%. These joints are assumed not to be completely free. The effect of temperature on load transfer was also examined. In September, load transfer was found to be 39% for the normal joints and 35% for the four free joints.

Laboratory Testing

Cores of the pavements were taken for the determination of layer thickness. Average thickness of the PCC was 91 mm (3.6 in.), and average of the HMA was 79 mm (3.1 in.). The properties of the PCC were elastic modulus of 23,442 MPa (3.4×10^6 psi), compressive strength of 33 MPa (4,800 psi), indirect tensile strength of 4.24 MPa (615 psi), and unit weight of 2137 kg/m^3 (133 lb/ft^3). The modulus and unit weight of the PCC are slightly lower than those for normal PCC. For the HMA, the resilient modulus was 5,247 MPa (761 ksi) at 25°C and 12,308 MPa (1.8 x 10^6 psi) at 5°C. Its indirect tensile strength was 1.01 MPa (146 psi), and its unit weight was 2147 kg/m^3 (133 lb/ ft^3).

Findings

The original researchers identified several findings as a result of the field testing of the six whitetopping slabs at the Spirit of St. Louis Airport:

1. The whitetopping overlay is only partially bonded to the HMA pavement underneath. The strain at the bottom of the PCC layer and the strain in the HMA are not equal.
2. The whitetopping slabs do not experience significant curling movements daily after construction.
3. Stresses at the center of the slabs and at the joints are comparable in the whitetopping slabs. When the joints are cut for a free edge, the stresses increase.
4. Load transfer at the whitetopping joints was measured. For normal joints, load transfer efficiency is 69%. For free edges, the load transfer decreases to 41%.

VIRGINIA

Since the 1980s, the FHWA has maintained several pavement test sections made up of HMA at its Accelerated Loading Facility (ALF). In 1998, eight lanes of UTW overlays were placed over a portion of these HMA pavements. The test sections, located at the Turner–Fairbank Highway Research Center in McLean, Virginia, presented an excellent opportunity for evaluation of a UTW pavement under controlled loading conditions. The experiment resembles a real-world application of UTW, because the underlying HMA pavements had been previously loaded. The new UTW overlays were placed at varying thicknesses, joint spacings, additions of fibers, and HMA base types (*141*). Figure B9 includes a schematic of the eight lanes, including the observed crack patterns at the end of loading.

During the summer of 2000, the performance of the UTW pavements was evaluated as part of a research effort sponsored

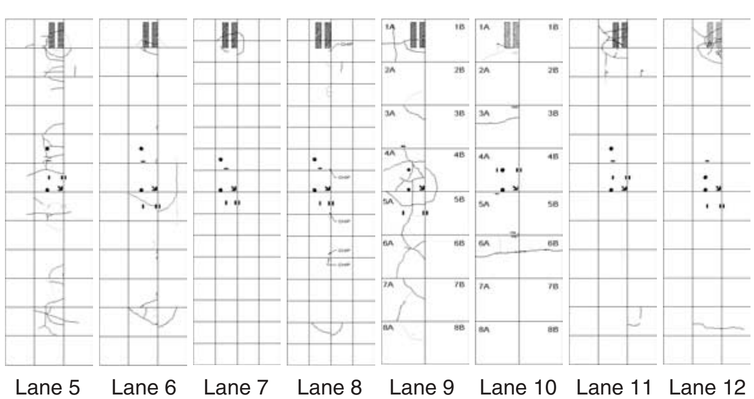


FIGURE B9 Schematic of UTW lanes at the FHWA ALF.

by the Innovative Pavement Research Foundation and the FHWA (*2*). As part of this evaluation, a number of observations were made, and as a result, hypotheses were developed as to the probable modes of failure.

Background

At the FHWA ALF, the eight UTW test lanes were constructed at dimensions of 3.7 m (12 ft) wide and 14.6 m (48 ft) long. Half of the lanes were constructed using fiber (polypropylene)-reinforced concrete and the other half with plain concrete. Two different UTW thicknesses and three different joint spacings were also constructed. Table B14 includes the test factorial used at the ALF.

The UTW was placed as an inlay—on top of HMA that had been milled so that the final UTW surface would match the existing grade. The underlying structure, before milling, consisted of 200 mm (8 in.) of HMA atop 460 mm (18 in.) of unbound crushed aggregate base atop 610 mm (24 in.) of AASHTO A-4 uniform subgrade. Each of the lanes (except for Lanes 6 and 10) was constructed with a different type of HMA. Before the UTW experiment, each of the HMA lanes was loaded with the ALF, and the permanent deformation (rutting) was recorded at periodic intervals. A summary of the HMA types and performance is provided in Table B15.

Before construction, resistance strain gauges were installed in the UTW along both the longitudinal and transverse directions at three different depths. For a typical lane, a total of nine longitudinal and three transverse strain gauges were installed. Three extra longitudinal strain gauges were installed in Lane 5. In addition, strain rosettes were installed at the pavement corners at two different depths. Linear variable displacement transducers were also installed to measure deflection at the slab center and at the joint. A schematic of the typical installation locations is given in Figure B10.

During the ALF loading, strain and deflection information was discretely sampled for every sensor at short increments of load applications (usually one set of readings per day of testing). The test load applied was set constant to 55 kN (12.3 kips), except for the first 310,000 load applications on Lane 12 where a load of 44 kN (10 kips) was used. The speed of the ALF was set constant at 16.6 kph (10.3 mph). Each unidirectional pass of the ALF has a duration of approximately 3 s. Sensor sampling rate was approximately one-thousandth (0.001) of a second.

Loading of the various lanes is illustrated in Figure B11. As can be seen, the number of load applications applied ranged from 200,000 to more than 1 million. Over the course of the loading, distresses such as cracking progressed and were monitored through periodic visual surveys of the conditions.

TABLE B14
LANE ASSIGNMENTS FOR UTW DESIGNS AT THE FHWA ALF

UTW Design Thickness [mm (in.)]	Joint Spacing [m (ft)]	Fiber Concrete	Plain Concrete
64 (2.5)	1.2 (4)	Lane 5	Lane 6
	0.9 (3)	Lane 7	Lane 8
89 (3.5)	1.8 (6)	Lane 9	Lane 10
	1.2 (4)	Lane 11	Lane 12

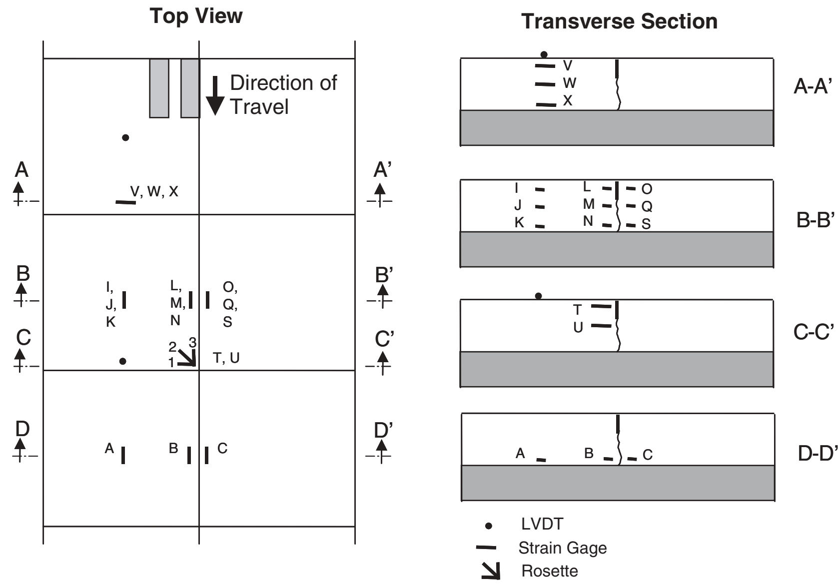
Typical Distresses

During the field visit, the project team observed a number of different structural distress types. The primary distress types observed included

- Mid-slab transverse cracking,
- Mid-slab longitudinal cracking,
- Corner cracking,
- Joint faulting, and
- Spalling.

TABLE B15
HMA CHARACTERISTICS AT THE FHWA ALF

Lane	Binder Type	Mix Type	ALF Wheel Passes for 20 mm Total Rut Depth	% Rut Depth in HMA at 20 mm Total Rut Depth
5	AC-10	Surface	1,160	82%
6	AC-20	Surface	1,790	97%
7	Styrelf	Surface	23,200	63%
8	Novophalt	Surface	39,600	32%
9	AC-5	Surface	480	85%
10	AC-20	Surface	1,790	97%
11	AC-5	Base	5,450	82%
12	AC-20	Base	22,100	85%



Note: Gauges A, B, and C were installed only in Lane 5. LVDT = linear variable displacement transducer.

FIGURE B10 Typical schematic of sensor locations.

Figure B12 illustrates a typical transverse crack found at the ALF UTW. Longitudinal cracks were observed to be of a similar appearance. Figure B13 shows a typical corner cracking. It should be noted that corner cracks were often observed to occur in tandem (roughly symmetrical about a transverse crack), as is illustrated here. Figure B14 is a photograph of the joint faulting distress. The most significant faulting was along the longitudinal joint, owing to the channelized nature of the loading, although some transverse faulting was observed as well. Finally, Figure B15 shows typical spalling. Most of the observed spalling was of low severity. In most cases, the spalled concrete has remained in place. In some of these figures, the distress has been graphically enhanced for clarity.

Summary of Observations

At the time of the field visit in 2000, Lanes 6 and 10 were being loaded by the ALF as part of another innovative research effort. However, observations were made of the remaining lanes as follows:

Lane 5

Features were as follows: 64-mm (2.5-in.) UTW design thickness—1.2 m (4 ft) panels—fibers
194,500 loads—HMA: AC-10 binder, surface mix.

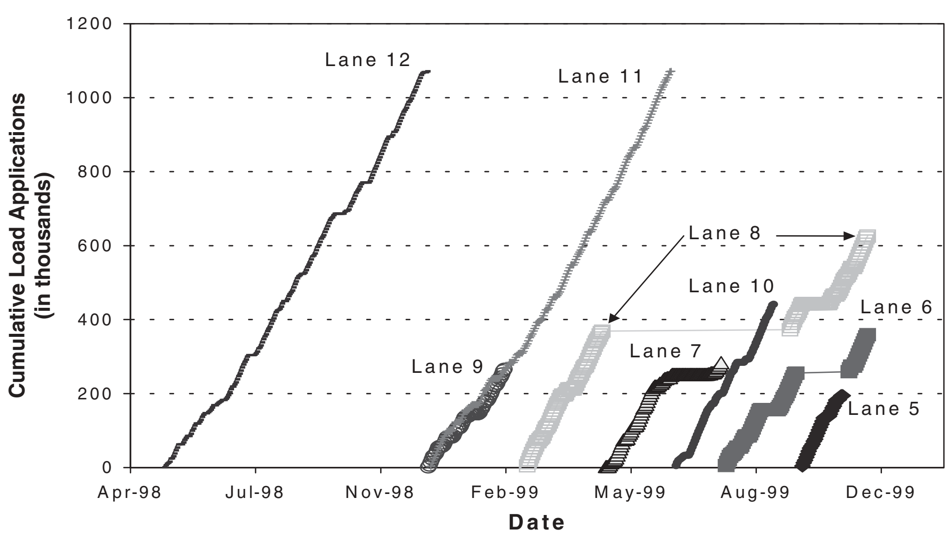


FIGURE B11 Loading history of the ALF UTW pavements.

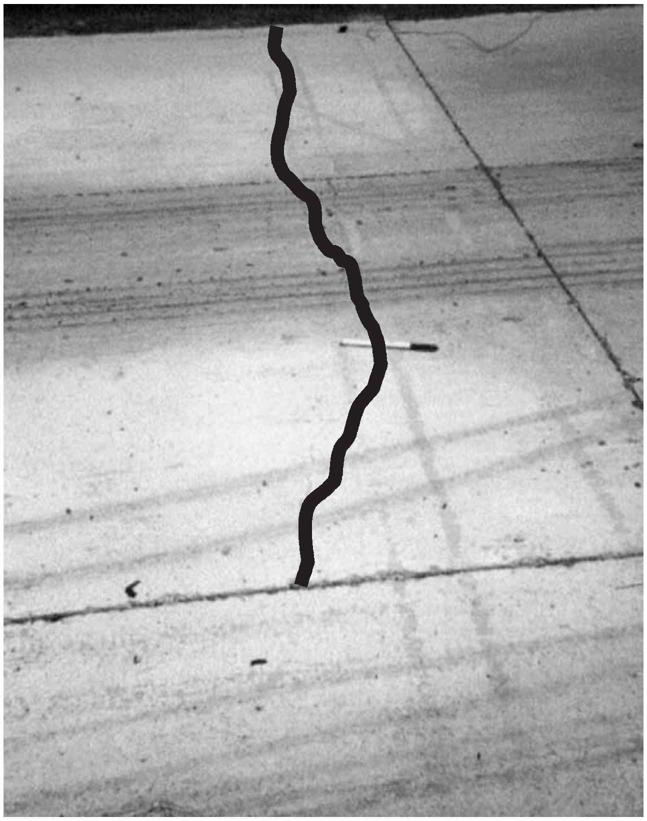


FIGURE B12 Typical UTW transverse cracking at the FHWA ALF.

The panels in this lane were heavily damaged. With the exception of the southernmost panel, every panel along the loaded column of slabs had some form of cracking. The majority of these panels demonstrated corner cracking. Three of the panels also had transverse cracking. A significant degree of joint faulting along the longitudinal joint adjacent to the load was also observed. In the panels adjacent to the loaded column of panels, some additional cracking was observed. It appeared that most of this cracking was reflected across the longitudinal joint from the center-loaded column of panels.

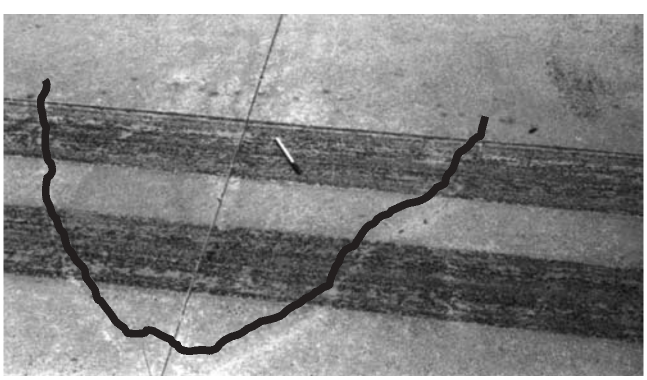


FIGURE B13 Typical UTW corner cracking at the FHWA ALF.

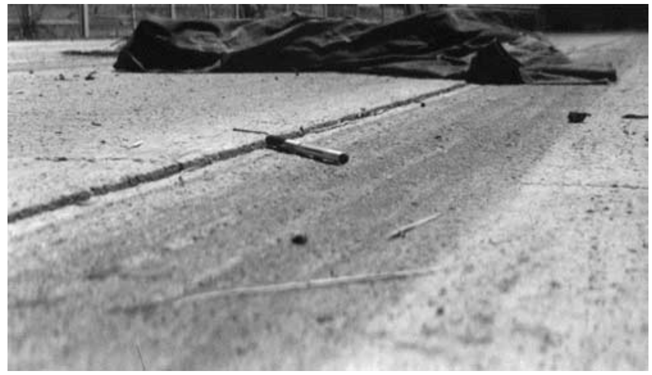


FIGURE B14 Typical UTW longitudinal faulting at the FHWA ALF.

Lane 7

Features were as follows: 64-mm (2.5-in.) UTW design thickness—0.9 m (3 ft) panels—fibers
283,492 loads—HMA: Styrelf binder, surface mix.

The panels in this lane appeared to have no observed distress. A few cracks were observed at the impact zone, where the wheel load first exerts an impact on the pavement at loading. No joint faulting was detected either. Overall, this lane appeared to be in excellent condition.

Lane 8

Features were as follows: 64-mm (2.5-in.) UTW design thickness—0.9 m (3 ft) panels—no fibers
625,838 loads—HMA: Novophalt binder, surface mix.

Lane 8 also appeared to be in very good condition. In addition to the cracking found at the impact zone, a few additional cracks were observed. They included tandem corner cracks on the southern end of the lane as well as a corner crack and a partial longitudinal crack on the north end of the lane. In addition to these cracks, five spalls were observed. Each of the spalls occurred at a corner and was approximately

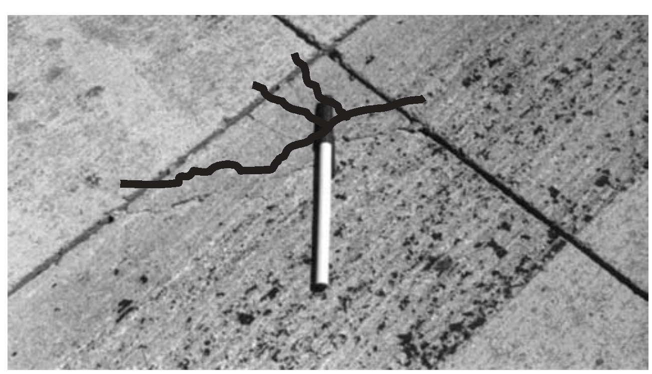


FIGURE B15 Typical UTW deflection spalling at the FHWA ALF.

25 to 50 mm (1 to 2 in.) in size. Lane 8 also was observed to have a minor degree of faulting along the longitudinal joint.

Lane 9

Features were as follows: 89-mm (3.5-in.) UTW design thickness—1.8 m (6 ft) panels—fibers
265,913 loads—HMA: AC-5 binder, surface mix.

Lane 9 appeared to be moderately to heavily damaged. Every panel in the loaded column of slabs was cracked. Corner cracks prevailed, with every cracked slab containing at least one of this type. The corner cracks appeared to occur in tandem, mirrored across a transverse joint. In one case, corner cracks occurred on four adjacent panels, mirrored across both the transverse and longitudinal joints. Half of the loaded column of slabs also had longitudinal cracking, and one transverse crack was found. A moderate degree of joint faulting was observed along the longitudinal joint.

Lane 11

Features were as follows: 89-mm (3.5-in.) UTW design thickness—1.2 m (4 ft) panels—fibers
1,071,302 loads—HMA: AC-5 binder, base mix.

Minimal damage was observed on this lane. With the exception of the impact zone, only three cracks were noted: one corner crack, one partial longitudinal crack, and one partial transverse crack. However, some joint faulting was observed.

Lane 12

Features were as follows: 89-mm (3.5-in.) UTW design thickness—1.2 m (4 ft) panels—no fibers
1,071,312 loads—HMA: AC-20 binder, base mix.

Lane 12 was also observed to be in good condition. One full and one partial transverse crack were noted on the southern end of the lane. No significant faulting was observed.

Of those structural distresses observed at the ALF UTW, corner cracking seemed to be the most prevalent, followed by transverse and some longitudinal cracking. Faulting of the longitudinal joint was also observed to be somewhat significant. However, it should be recognized that the channelized nature of the ALF loading on the UTW pavements does not indicate in-service pavements that are subject to vehicle wander. The type of longitudinal faulting observed here would probably not occur to the same degree in those cases. Transverse cracking was also found to some degree, as well as some longitudinal cracking. Finally, spalling was found to exist on some of the lanes.

Pavement Distress Mechanisms

Both during and after the field visit, a review was made of the various types of information collected at the ALF. Included were data on the UTW as well as the underlying HMA support layers, including strain, deflection, and temperature data collected during the accelerated loading. In addition, laboratory test data, climatic information, and design and construction records were reviewed. Hypotheses were drafted on the various failure mechanisms occurring at the ALF by comparing results of the observed UTW pavement performance with the available information.

A relationship between the observed pavement distress levels and the nature of the underlying HMA was realized from this analysis. Before the construction of the UTW at the ALF, the underlying HMA pavements were loaded by the ALF device as part of an experiment, to characterize permanent deformation (rutting) of the different mixes. Figure B16 illustrates the rutting of the HMA material used as the support layer for the UTW.

To make a rough quantitative comparison, the degree of cracking observed on each of the lanes was quantified with the use of a cracking index. The cracking index is a weighted average of the number of cracks observed on each lane. The index is a summation of the number of cracks per lane, with

FIGURE B16 Typical HMA rutting at the FHWA ALF.

a full-panel crack assigned a value of 1.0, a partial-panel crack of 0.5, and a small chip or break of 0.1. That given value is then compared with the rutting that is quantified as the number of ALF load applications to reach 20 mm (0.8 in.) of total rutting. The results of this comparison are shown in Figure B17. Although not given here, a similar relationship was found between the rates of change of ride quality for each lane compared with the rutting susceptibility of the underlying HMA (*49*).

On the basis of these observations, the following sections describe hypotheses developing in regard to the nature of the various structural distresses observed at the ALF UTW.

Corner Cracking

As mentioned, corner cracking was the most commonly observed structural distress at the ALF UTW. This distress is caused when the fatigue limit is exceeded in the concrete material. The fatigue limit is commonly defined as a function of the stress-to-strength ratio. In most of the lanes, the strength of the concrete over the period of loading remained approximately constant. However, the stress state in the concrete most likely changed with the number of load applications. The source of such a finding is believed to be the result of a change in the support conditions owing to permanent deformation of the support layers. That effect is shown in Figures B18 and B19. In Figure B18, the action of repeated loading across the slabs leads to a permanent deformation of the HMA beneath the UTW. This void, in turn, causes a cantilever effect that increases the stresses in the concrete at the top surface. Once the fatigue limit is reached in the concrete, the UTW failure comes in the form of a corner crack. This phenomenon is shown in Figure B19.

It should be emphasized that if cores were to be removed from the edge, the PCC and HMA materials may appear to be full intact (bonded). A combination of residual tensile stresses, microstructural damage (microcracking), and/or a loss in density of the HMA in proximity to the interface is possible. The result is a weakened area that may lead to a loss of support, even if the interface is not visibly debonded. Therefore, an expression such as "virtual void" may be more appropriate in describing the suspected loss of support.

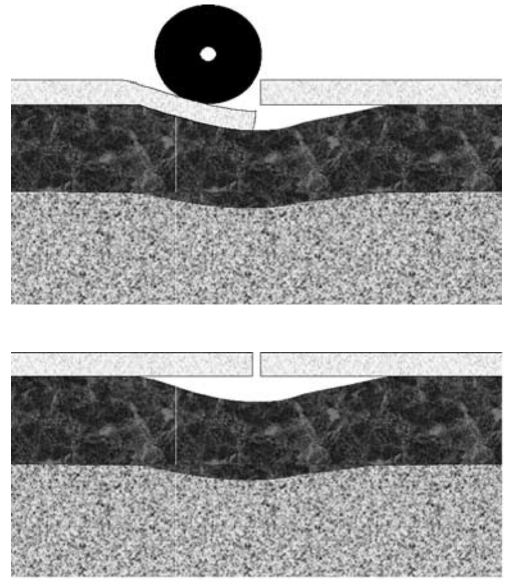


FIGURE B18 Permanent deformation of an HMA base.

Another possible explanation for the corner cracking is that the cracks may have been initiated on the loaded (longitudinal edge) of the lane and propagated diagonally toward the closest intersecting joint with each successive wheel load. This hypothesis is derived from the belief that changes to the stress state are induced by the moving wheel load in the slabs.

Mid-Slab Cracking

As with corner cracking, mid-slab cracking is caused when the concrete loading exceeds the fatigue limit. As illustrated in Figure B20, one hypothesis is that the mid-slab cracking is initiated at the bottom of the slab. As the wheel load passes directly over the mid-slab, the stresses are highest directly

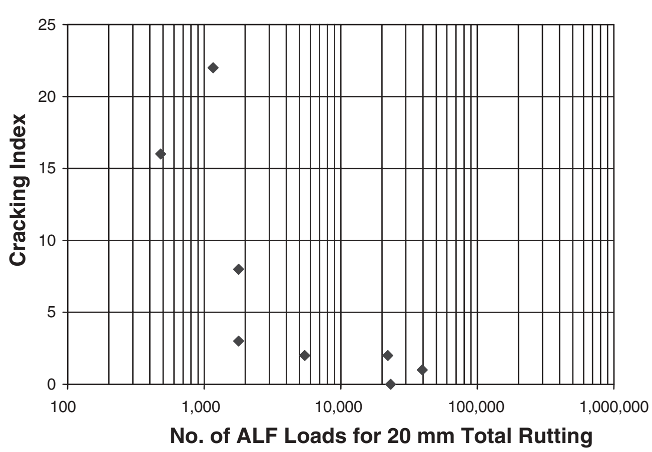


FIGURE B17 Comparison of HMA rutting to UTW cracking.

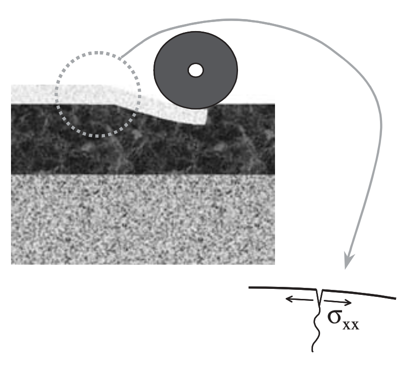


FIGURE B19 Corner cracking mechanism in a UTW slab.

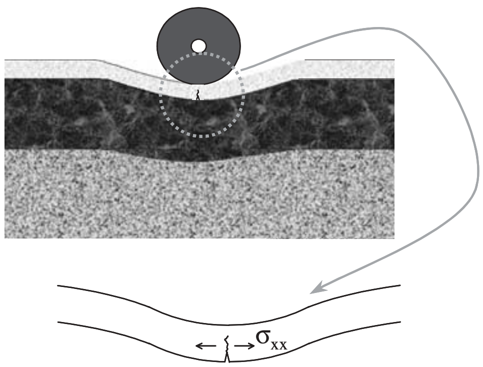


FIGURE B20 Mid-slab cracking mechanism in a UTW slab.

beneath the load at the edge. These stresses can be further compounded by the presence of a void, or soft area, beneath the slab in the support layer system.

A second hypothesis is that the cracks are initiated at the top of the slab, induced by tensile stresses at the top as the wheel load rolls onto the slab in question. Strain gauges instrumented in the slab verify a stress reversal in the top of the slab, indicating this effect, as shown in Figure B21.

Joint Faulting

Faulting was observed along both the longitudinal and transverse joints at the ALF UTW. For the longitudinal joints, the mechanism is unique owing to the channelized nature of the wheel loading. Although the ALF has the ability to test with transverse wheel wander, during the testing of the UTW pavements, the wheel loads were constrained along the same line. As a result, the high vertical stresses introduced into the support layers resulted in permanent deformation. This effect can be seen in Figure B22.

Figure B23 illustrates the hypothesized mechanism for the transverse faulting also observed at the ALF UTW. Under wheel loading, both vertical and shearing forces were introduced into the support layer materials. These forces resulted in a vertical and shear deformation of these layers, which, under repeated loading, caused a transverse fault along the joint.

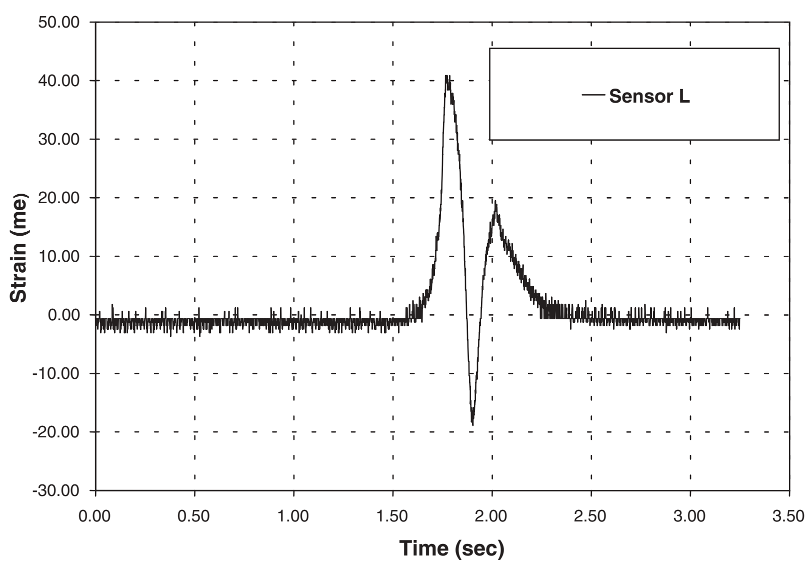
Joint Spalling

There exist two common types of joint spalling. The first type, sometimes termed "delamination spalling," is caused by the combined effect of horizontal microcracking introduced during the early-age concrete construction and traffic loading that eventually weakens the horizontal crack. The result is a flat-bottom spall. The second common type of spalling is sometimes termed "deflection spalling." This type of spalling is common on airport pavements where high deflections in the slabs cause a localized crushing of the material at the joints. Owing to the thin geometry of the UTW slabs (with respect to the heavy loading), deflection spalling is believed to be the governing mechanism. Figure B24 illustrates this phenomenon.

Findings

Various failure mechanisms at the ALF UTW have been identified and summarized in this appendix. From observations made after the experimental loading was completed, along with a review of the project information, hypotheses were formed and documented on the various mechanisms leading to the observed distress types.

It is concluded that a common element of each of the observed distress types is the permanent deformation characteristics of the support layers. The UTW pavements that



Note: Sensor is from Lane 5. Refer to Figure 2 for sensor location. (+) = tension.

FIGURE B21 Measured strain response of a top mid-slab longitudinal gauge.

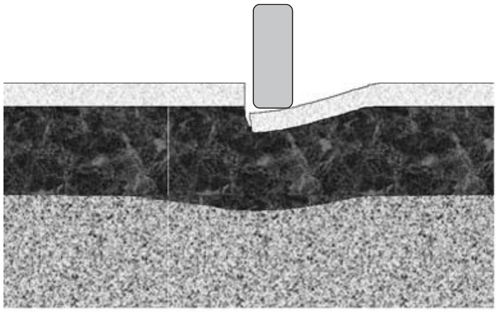


FIGURE B22 Longitudinal joint faulting mechanism in ALF UTW slabs.

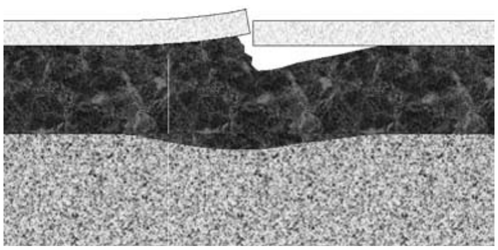


FIGURE B23 Transverse joint faulting mechanism in ALF UTW slabs.

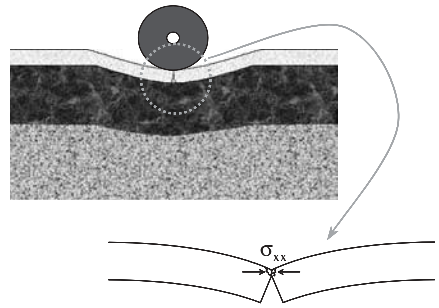


FIGURE B24 Joint spalling mechanism in ALF.

were constructed on the softer HMA sections (those more prone to rutting) performed more poorly than did those constructed on stiffer HMA materials.

It is also worth noting that the ALF loading in this experiment was conducted in a channelized manner along the slab edge. In practice, the loading of the slabs will vary from slab edge to center. However, the slab edge loading is believed to be the critical position, and the majority of design procedures are based on this assumption.

Abbreviations used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ITE	Institute of Transportation Engineers
NCHRP	National Cooperative Highway Research Program
NCTRP	National Cooperative Transit Research and Development Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
SAE	Society of Automotive Engineers
TCRP	Transit Cooperative Research Program
TRB	Transportation Research Board
U.S.DOT	United States Department of Transportation