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**DRAINAGE EVALUATION
AT THE U.S. 50 JOINT SEALANT EXPERIMENT**

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Division of Research and Advanced Studies
of the University of Cincinnati

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ABSTRACT

This Thesis presents an investigation of the subsurface drainage features of the test pavement at the United States (U.S.) Route 50 joint sealant experiment near Athens, Ohio. The pavement incorporates a 4-in. thick open-graded base, whose design is assessed using the software *DRIP 2.0*, distributed by the Federal Highway Administration. It is found that the specified base thickness and permeability combination do not meet federal guidelines, evidently because no design calculations had been performed prior to construction. A field inspection of the test pavement drainage features revealed that these had received very scant to no maintenance. Drainage outlets were unmarked, overgrown by vegetation, difficult to find or sometimes missing, often clogged and occasionally damaged. A literature review indicated that such problems are endemic in many states and point to an under appreciation of proper drainage in concrete pavements. A review of the structural performance of the test pavement found no obvious correlation with the condition of a variety of sealant treatments applied during the experiment.

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LIST OF ABBREVIATIONS

AASHO	American Association of State Highway Officials
AC	Asphalt Concrete
ATB	Asphalt Treated Base
ATPB	Asphalt Treated Permeable Base
<i>b</i>	Total width of the pavement
B.C.	Before Christ
<i>c</i>	Horizontal distance from the edge of the pavement to the edgedrains
C	Celsius
Caltrans	California Department of Transportation
CRCP	Continuously Reinforced Concrete Pavements
CTPB	Cement Treated Permeable Base
C_u	Coefficient of Uniformity
D_{15}	Particle Size Diameter Percentile (15% passes this size)
D_{50}	Particle Size Diameter Percentile (50% passes this size)
D_{85}	Particle Size Diameter Percentile (85% passes this size)
d	day
D	Edgedrain Diameter
DRIP	Drainage Requirements in Pavements
EB	Eastbound
EBNV99	November 1999 sealant evaluation in the eastbound lanes

EBMR00	March 2000 sealant evaluation in the eastbound lanes
EBOC00	October 2000 sealant evaluation in the eastbound lanes
EBJN01	June 2001 sealant evaluation in the eastbound lanes
EBOC01	October 2001 sealant evaluation in the eastbound lanes
ESAL	Equivalent Single Axle Load
F	Fahrenheit
FHWA	Federal Highway Administration
ft	feet
GPS	General Pavement Studies
H	Permeable Base Thickness
H _{min}	Minimum Permeable Base Thickness
hr	hour
IA	Iowa
I _c	Crack Infiltration Rate
IDOT	Illinois Department of Transportation
in.	inches
IRI	International Roughness Index
IRIbh	International Roughness Index, both wheel tracks
IRIlf	International Roughness Index, left wheel tracks
IRIrt	International Roughness Index, right wheel tracks
JPCP	Jointed Plain Concrete Pavement
JRCP	Jointed Reinforced Concrete Pavement

k	Coefficient of Permeability
km	kilometer
L_R	Longest flow path of permeable base
LCB	Lean Concrete Base
LT	Left Offset
LTPP	Long Term Pavement Performance
m	meters
MAYS	Mays Number
mm	millimeters
n	Roughness Coefficient
n_e	Effective Porosity
NCHRP	National Cooperative Highway Research Program
NJ	New Jersey
No.	Number
ODOT	Ohio Department of Transportation
PCC	Portland Cement Concrete
PDI	Pavement Distress Index
PennDot	Pennsylvania Department of Transportation
PSI	Present Serviceability Index
q_i	Pavement Infiltration
RPPR	Rigid Pavements Performance and Rehabilitation
RT	Right Offset

S	Longitudinal Gradient
SHRP	Strategic Highway Research Program
SL	Self Leveling
SPS	Specific Pavement Studies
S_R	Resultant slope of permeable base
S_x	Cross Slope
UC	University of Cincinnati
U.S.	United States
W	Required permeable base width
WASHO	Western Association of State Highway Officials
WB	Westbound
WBNV99	November 1999 sealant evaluation in the westbound lanes
WBMR00	March 2000 sealant evaluation in the westbound lanes
WBOC00	October 2000 sealant evaluation in the westbound lanes
WBJN01	June 2001 sealant evaluation in the westbound lanes
WBOC01	October 2001 sealant evaluation in the westbound lanes
WisDOT	Wisconsin Department of Transportation

1 INTRODUCTION

1.1 Introduction

The construction of pavement subsurface drainage features has increased significantly in the past three decades due to new awareness brought about by publications by Cedergren (1974, 1988) and Moulton (1980). Some of these features include permeable bases along with edge drains. Although their popularity has increased, it is argued by some that their performance (especially that of permeable bases) has not lived up to the hype.

This thesis focuses on the effects poor drainage design can have on a highway, using as an example the drainage features at the United States (U.S.) Route 50 joint sealant test site near Athens, Ohio. It is shown that the overall performance of permeable bases is excellent and that any disappointing experiences may be the result of insufficient outlet drain maintenance, pavement structural stability, base thickness and gradation design, or a combination thereof.

1.2 U.S. 50 Joint Sealant Experiment

This thesis came to life during the U.S. 50 sealant experiment in Athens, Ohio that the University of Cincinnati (UC) research team conducted for the Ohio Department of Transportation (ODOT) between 1997 and 2001. The sealant experiment involved the installation of various joint sealants in the transverse joints of a newly constructed Portland cement concrete (PCC) pavement. The experimental design for this project was

developed in 1997 by the Federal Highway Administration (FHWA) and ODOT to provide data for the evaluation of the performance of various joint seals and joint configurations. Fifteen combinations of materials and joint configurations are used in the experiment, which includes unsealed control sections. The purpose of these pavement test sections, located in the Wet-Freeze climatic zone, is to duplicate and complement similar sections constructed in other states under the Strategic Highway Research Program (SHRP) Specific Pavement Studies (SPS)-4 experiment. The test pavement is divided into fifteen test sections, each section typically being 183 m (600 ft) in length, but also includes some longer sections. Each test section incorporates about thirty joints. In accordance with the experimental design, two replicates of each of fifteen chosen material-joint configuration combinations are provided. Two of these combinations involve unsealed joints. In each case, one replicate is in the eastbound lanes, built during the 1997-98 construction season, and the other in the westbound lanes, placed during the 1998-99 construction season.

During an evaluation of the highway for the sealant experiment in Spring 2000, the UC research team noticed premature cracking within much of the pavement, at the same time that persistent rains had flooded the ditches along the side of the highway; the research team began to postulate that the two were perhaps related. After the evaluation, a literature survey was conducted and it was resolved that the drainage features of the highway, specifically the subsurface features, should be evaluated during the subsequent visit in October, 2000. Those drainage features were found in poor condition and the research team collected enough data and evidence to proceed with this thesis.

1.3 Project Objectives

The specific objectives of this thesis are as follows:

1. To determine the adequacy of the subsurface drainage design incorporated at the project site;
2. To demonstrate how a lack of maintenance can affect the subsurface drainage system;
3. To analyze the results from the most recent sealant evaluation so as to identify those treatments that are most effective; and
4. To investigate if a correlation exists between sealant treatment and pavement performance.

1.4 Thesis Organization

This thesis presents an introduction to the important subject of subsurface drainage in pavements and a brief history of the topic, and outlines the effects poor drainage design can have on a highway. Chapter 2 provides a literature review of subsurface drainage, discussing its history and national record to date. A field evaluation of the drainage features at the U.S. 50 joint sealant test site near Athens, Ohio is then documented in Chapter 3, along with a series of design calculations for this subsurface drainage system. Chapter 4 presents the most recent results from the U.S. 50 sealant experiment pertaining to both the sealant treatments and to the overall pavement condition to date. Trends in sealant performance are examined and the effectiveness of each material and joint configuration to date is summarized. Finally, Chapter 5

summarizes the outcomes of this study and provides a list of recommendations for future investigations.

2 LITERATURE SURVEY

2.1 Introduction

Water is always present in the pavement subsurface, generally in the form of free water, capillary water, bound moisture, and water vapor. Among these, capillary water, bound moisture, and water vapor are found only in limited amounts, and are not significant factors in the development of pavement distress. Free water, on the other hand, is the cause of great concern for pavement engineers, not only because it is the most abundant, but also because it can decrease the strength of the subsurface layers in three ways:

1. By reducing the apparent cohesion by lowering the capillary forces;
2. By reducing the friction by reducing the effective weight of the materials below the water table; and
3. By generating excess or oscillating pore water pressures.

Excess pore water pressures manifest themselves directly on the surface of the pavement in a phenomenon called pumping. High pavement deflections generate oscillating pore water pressures in water-laden, erodible silt-sized materials, thereby causing fine particles to be ejected through joints and cracks. The surface slab may then suffer appreciable loss of support, especially if this process is permitted to occur repeatedly. Pumping may be prevented by techniques that guard against high deflections and erodibility, but equally effectively by those that address the presence of excess water, e.g., by sealing the joints and by providing adequate subsurface drainage. Joint sealing

creates an impermeable barrier at the surface that reduces the amount of water entering the subsurface, while adequate drainage prevents excess pore pressures from developing. Excess pore water pressures can diminish the load transfer abilities of the base layers and may cause higher than expected subgrade stresses. In cold weather, problems may be compounded by the onset of another detrimental phenomenon, namely freeze-thaw.

2.2 Historical Background

The need for proper drainage has been known for millennia, and as early as 300 B.C. roads were constructed at higher elevations to avoid the effects of water. John Loudon McAdam (1756-1836), commonly referred to as the grandfather of modern pavement engineering, also realized the importance of good drainage when he addressed the London Board of Agriculture in 1820. He stated, “if water pass through a road and fill the native soil, the road whatever may be its thickness loses support and goes to pieces.” McAdam also demonstrated that those roads in Great Britain that did not account for the effects of water had “produced most of the defects” (McAdam, 1820).

The effects of poor subgrade drainage were also noted at the Bates Road Test in the early 1920s (Older, 1924), when demand for better roadways was growing due to the evolution of the automobile. Studies such as those by Izzard (1944) and Barber and Sawyer (1952) established the fundamentals of pavement drainage analysis, and identified the critical parameters involved, notably, soil and aggregate permeability and roadway gradient. Over time, however, a shift in design philosophy occurred from drainability to stability. The 1940s and 50s saw the construction of several major road

tests [Maryland, Western Association of State Highway Officials (WASHO), and American Association of State Highway Officials (AASHO)], but none of these tests incorporated any type of drainage features. The detrimental effects of water were apparent as the roadways began to show distress during a short time immediately after the wet spring season, yet little consideration was given to drainage in the 1961-1962 AASHO Interim Design Guides. Drainage features, such as permeable bases, were gradually replaced with thicker and denser bases, particularly those that are more resistant to erosion. This may be in part due to the development of subgrade and base material treatments that can render such materials insensitive to moisture. More importantly, advances in the structural analysis of pavements have encouraged engineers to make bases thicker and stiffer, and to ignore the detrimental effects of moisture related damage even though full saturation levels might be expected.

At about the same time the AASHO guide was being published, Lovering (1960) proposed a two-layer subsurface drainage system. The top layer consisted of a highly permeable open-graded aggregate; the bottom was a filter layer designed to prevent fines from clogging the overlying permeable layer. The concern of high cost, construction, and stability of the system, however, caused a lack of interest in the idea. To address the instability of the permeable base, Lovering and Cedergren (1962) proposed adding 2-3% asphalt that would bind the aggregate together and create enough strength to support construction traffic. The addition of asphalt actually increases the permeability of the permeable base because it requires fewer fines for strength. In 1967, the first highway section was constructed containing an asphalt treated permeable base (ATPB) in

California. This test section was successful and prompted the construction and study of more pavements containing permeable bases (Forsyth, 1991).

In 1970, the Federal Highway Administration (FHWA) sponsored Harry Cedergren and Ken O'Brien and Associates of California to study the water related problems in pavements. Their publication (Cedergren, et al. 1973) was the first design guide for subsurface drainage in pavements and is considered a classic among pavement engineers ushering a new era of interest in and attention to subsurface drainage. The authors proposed a subsurface drainage system that contained a highly permeable open-graded base, a filter or separator layer, parallel collector pipes (edge drains), transverse outlets to remove the water from the pavement, and markers to clearly show the location of the outlets. Proper maintenance of the subsurface drainage system is, of course, essential to maintaining its drainage capabilities, even if the system is designed and constructed properly (Moulton, 1980).

The state-of-the-practice in the area of pavement subsurface drainage systems has been summarized in two more recent, complementary publications from the National Cooperative Highway Research Program (NCHRP) (Ridgeway, 1982; Christopher and McGuffey, 1997). The investigators noted that although “many premature pavement failures (occurring at less than 50 percent of expected life) have been traced to inadequate subsurface drainage, different philosophies exist on how to reduce the effects of this problem.” Differences in pavement drainage practices were attributed to “inconsistencies in the reported performance of pavements with drainage systems,” even though “inadequate performance of pavements with drainage systems appears to be related more

to inconsistencies in design, construction, and maintenance than in the philosophy of positive pavement drainage.” The NCHRP studies also emphasized that “poor construction techniques can destroy the best-designed subsurface drainage system,” and that “maintenance of subsurface drainage systems is essential to the long-term success of the drain system and, subsequently, the pavement.” They concluded that “a preponderance of evidence was found supporting the philosophy that a combination of good sealing and good drainage, with a commitment to long-term maintenance, will lead to the optimum performance of a pavement system.”

2.3 Components of Subsurface Drainage

Excellent discussions of the components of a subsurface drainage system and summaries of their requirements are provided in several worthwhile publications, including those in FHWA (1992) and NCHRP *Synthesis of Highway Practice 239* (Christopher and McGuffey, 1997). The most pertinent such information is presented below.

2.3.1 Permeable Base

There is no standard grain size distribution for drainage layers as different state agencies use different gradations. The permeable base can be designed with a cement binder to add stability; typically, asphalt is used but Portland cement may also be employed. The ability to evacuate water expeditiously diminishes significantly if an open-graded base becomes clogged with subgrade fines migrating upward, thereby

resulting in increased excess pore water pressures and decreased shear strength in the supporting layers. The coefficient of permeability of permeable bases has been shown to be reduced by nearly 100% when clogged with soil fines (Tan, et al., 2003).

2.3.2 Separation Layer

A separation or filter layer may be used beneath a permeable base to prevent the upward migration of soil fines that may cause clogging. Geotextiles may sometimes replace soil-aggregate mixtures in this function.

The FHWA (1992) suggest that the maximum percent of fines in aggregate separator layers should not exceed 12% and the coefficient of uniformity (C_u) should be between 20 and 40. In addition, compatibility requirements must be met between the subgrade and separator layer, and separator layer and permeable base. The FHWA (1992) suggest the following, in which D_i is the particle size diameter percentile:

Separator/subgrade: $D_{15} \text{ (separator layer)} \leq 5 D_{85} \text{ (subgrade)}$

$D_{50} \text{ (separator layer)} \leq 25 D_{50} \text{ (subgrade)}$

Separator/permeable base: $D_{15} \text{ (base)} \leq 5 D_{85} \text{ (separator layer)}$

$D_{50} \text{ (base)} \leq 25 D_{50} \text{ (separator layer)}$.

2.3.3 Edge drains

Edge drains are usually made of perforated corrugated plastic pipe and run parallel to the highway. They collect water from the permeable base and relay it to the

drainage outlets. It is important that these drains be kept at some minimum slope to prevent fines from clogging them.

2.3.4 Collector Pipes

Collector pipes are placed perpendicular to the highway to allow water to escape from the edge drains to the outlets. The FHWA suggest pipes be placed at “convenient intervals” so that water can be discharged at appropriate locations (Moulton, 1980), but no maximum spacing value is specified. Mallela, et al. (2000) suggest non-perforated metal or smooth rigid pipes to allow for a higher discharge velocity so that fines will not be allowed to settle and collect in the pipe.

2.3.5 Outlets

Outlets are crucial features of the subdrainage system since they are located at the end of the drainage path and consequently require the most maintenance. This is often ignored, allowing them to be overcome by vegetation and silt. Outlets need to be cleared regularly of any impairment, such as silt, vegetation, and rodent nests. Metal screens are typically inserted to keep rodents out, but they sometimes prove ineffective and even counterproductive, since they collect debris and prevent water flow (Steffes, et al., 1991). Outlets should also be protected from mowing machine damage. To accomplish this, most state agencies use pre-cast concrete outlets to prevent them being crushed. The FHWA also suggest using markers to facilitate locating outlets for routine maintenance (Moulton, 1980).

2.3.6 Drainage Channels

Water is transported from the collector pipes to longitudinal drainage channels that distribute the water to the nearest stream or river. Drainage channels require hydraulic gradients large enough to prevent water from ponding and seeping back into the ground, thereby causing pavement distress (Birgisson and Ruth, 2003). Mallela, et al. (2000) suggest a minimum longitudinal gradient of 0.005 m/m (0.005 ft/ft). Proper maintenance, such as mowing and clearing, is also essential.

2.4 Maintenance

Pavement systems must have periodic maintenance if the design life is to be achieved. The application of pavement maintenance applies also to the subsurface drainage system, even if its extent is as humble as periodic mowing. Moulton (1980) states that proper maintenance to the subsurface drainage system is necessary for maintaining its capabilities even for systems designed and constructed properly. Similarly, the more recent NCHRP Synthesis of Highway Practice 285 notes that “there is a significant cost in terms of poor performing pavements to agencies that use edge drains and do not have an effective preventive maintenance program,” but that evidently “there is an apparent disconnect between maintenance, design and construction in many state agencies” (Christopher, 1997). Moulton (1980) also suggests that the department responsible for maintenance of the subsurface drainage system should keep detailed drawings of the drainage components so they may be found quickly and accurately; this holds especially true for drain outlets. Stivers, et al. (1999) provide excellent

standardized forms that may be used by inspection teams. It is important to keep records of all inspections in an efficient manner so that continuing maintenance problems can be quickly identified, lest they develop into more severe and expensive problems.

Specific maintenance requirements in each case can only be established through regular monitoring. Mallela, et al. (2001) and Stivers, et al. (1999) provide excellent quality assurance programs to track specific maintenance requirements. Some of these requirements are described in the following sections.

2.4.1 Underdrains

Underdrains include subsurface edge drains and collector pipes. Even with proper filter design, soil fines may accumulate and retard the flow of water after an extended period of time. Inadequate pipe gradients, uneven settlement of the system, and heavy sediment load are factors that aid clogging. Underdrains should be periodically cleaned or flushed to keep the flow rate at the desired level. Clean-out boxes or risers should be installed to accomplish this more quickly and easily. Studies have shown that occasional flushings with large quantities of clean water can restore drainage capacity. If flushing is not feasible or desirable, a long bendable device similar to a plumber's snake may be used to remove silt and debris.

2.4.2 Outlets

A regular monitoring program is crucial in discovering the maintenance needs for drain outlets. Cedergren (1974) suggested inspecting outlets at least once every three

months, which should not stress the budget or schedule of any department since the amount of effort involved is minimal. Outlets may become blocked due to weed growth, rodent nests, siltation of the adjacent ditch, debris from the roadway, inadequate slope, and human activity. Routine mowing can easily remove weed growth, but care should be taken not to damage the outlets in the process. Outlet markers should be installed and kept in good condition so that the outlets can be found easily and quickly, and their accidental damage can be avoided. If markers are damaged or destroyed they should be replaced or repaired immediately so that the outlet location is not lost. Markers should be tall enough so that they will not be obscured by tall vegetation.

2.4.3 Indirect Maintenance

Regular maintenance to other pavement components may contribute to the well-being of the subsurface pavement system. Such indirect maintenance tasks may include resurfacing the pavement and pavement shoulder, improving surface drainage systems, ice or snow control and removal, and resealing joints and cracks.

2.4.4 Recordkeeping

Moulton (1980) suggests that the department responsible for maintenance of the subsurface drainage system should keep detailed record drawings of the drainage components so they may be found quickly and accurately; this holds especially true for drain outlets. Stivers, et al. (1999) provide excellent standardized forms that may be used by inspection teams. It is important to keep records of all inspections in a well-organized

manner so that routine maintenance problems can be quickly identified and addressed, so that they do not develop into larger and more expensive problems.

2.5 Cost

Many state agencies shy away from subsurface drainage because of excessive costs due to additional materials and labor, but these additional costs are returned in the prolonged service life of the pavement. When compared to their undrained counterparts, drained rigid and flexible pavements exhibit annual savings of 41 and 21%, respectively, over their respective service lives (Forsyth, et al., 1987). Cedergren (1988) conservatively estimates that the damage rate in undrained pavements is 15 times higher than in drained sections. Based on these and other calculations, Cedergren concludes that from 1976 to 1990 government agencies could have saved two-thirds of the \$329 billion dollars expended on repairs. Mallela, et al. (2000) estimate that a well maintained drained pavement may cost 114 to 124% more than a traditional non-drained one. Nonetheless, they caution that it is essential to evaluate the increased expenditure over the asset's design life. In a hypothetical comparison between two pavement sections, one drained and one not, the investigators found the undrained section would suffered greater distress at the end of its design life than the drained section; in such cases the additional costs are justified.

2.6 Popularity of Permeable Base

In a survey conducted in 1985, 24 of the 49 state agencies that responded to a questionnaire indicated that they had used or did use permeable bases. The majority of these states had been using permeable bases for fewer than five years. Eleven states had more than five years experience with permeable bases, including one state agency with over 1,000 lane miles. The performance of permeable bases was positive since 78% of the agencies responded that their experience was either “good or excellent” (Baldwin, 1987). In 1994, another survey was conducted in which 19 of the 42 state agencies that responded stated that permeable bases were standard in Portland cement concrete (PCC) pavements and 77% at least occasionally used permeable bases (Christopher and McGuffey, 1997). As of 2003, nearly all state agencies had at least some pavements constructed with permeable bases (Hall and Correa, 2003).

2.7 Performance of Permeable Base

It is evident that the popularity of permeable bases has increased significantly in the past 30 years, and performance data presented in the literature have been for the most part rather favorable. Some of this information is presented below, organized alphabetically on a state-by-state basis.

2.7.1 California

The California Department of Transportation (Caltrans) has experimented with permeable bases since the late 1960s, and “has used asphalt treated permeable base

as a standard component in new pavement designs since 1983.” Such bases are found to increase “pavement life by increasing the pavement’s resistance to fatigue cracking failure” (Bejerano, et al. 2003). The use of either cement- or asphalt-treated permeable bases is allowed, but a lean concrete base (LCB) is mandated for heavy truck traffic roadways. Forsyth, et al. (1987) observed a 2.4:1 ratio of new crack formation in undrained to drained pavements. In one roadway, the undrained section exhibited nearly 50% slabs cracked, whereas the section containing a cement treated permeable base (CTPB) had no cracks whatsoever. These researchers also reported a 33 and 50% extension of service life in asphalt concrete and PCC pavements, respectively, that incorporated drainage layers. In another study, two highway sections built with permeable bases exhibited no cracking after approximately eight years of service, whereas two undrained sections had 18 and 47% cracking, respectively. A third, 25-year old section, constructed in 1965, had only 5% cracking, while the undrained section had experienced 10% cracking. California reported increases of 33 and 50% in minimum service life for asphalt and PCC pavements, respectively (Mathis, 1989). The optimistic outlook in California, however, was not long lived. In 1991, an inspection of an ATPB found it to be in a loose and unbound condition. Cores showed that migration and crushing of the aggregate had occurred and created voids in the ATPB. Faulting measurements taken on the same roadway, on the other hand, showed no significant difference in PCC slabs constructed over ATPB and LCB, yet after these findings Caltrans stated that “serious problems may exist with ATPB and PCC pavements” and

declared that “the base of choice under PCC pavements should be LCB” (Wells, 1991). Currently Caltrans is no longer constructing permeable bases (Caltrans, 2006).

2.7.2 Colorado

Early evaluations at the SPS-2 test site compared the performance of permeable bases to that of dense-graded bases. In the relatively dry climate of Colorado, no difference could be seen to date between the performance of the permeable base sections and those constructed over dense-graded bases (Suthahar, et al., 2000).

2.7.3 Illinois

The Illinois Department of Transportation (IDOT) observed premature transverse cracks, patches, and high deflections after evaluating continuously reinforced concrete pavements (CRCP) constructed with permeable bases. Heckel (1997) recommended that permeable bases should no longer be used in CRCP, even though “no firm conclusions can be reached regarding the cause of the problems.” Currently IDOT is no longer constructing permeable bases (IDOT, 2002).

2.7.4 Iowa

Iowa’s experience showed that the support for undrained pavements declined after approximately five years, whereas support for pavements incorporating drainage layers endured. They also found that undrained pavements had significantly more cracks than drained pavements (Mathis, 1989).

2.7.5 Kentucky

Kentucky has been experimenting with permeable bases since 1978. After thirteen years of observation, Sharpe (1991) stated that “the permeable bases are still in good condition.”

2.7.6 Michigan

Surveys conducted on three of Michigan’s permeable base sections found no faulting or cracking, and less apparent D-cracking than in sections built over dense-graded bases. Areas of pumping and longitudinal joint spalling were found in the latter (Mathis, 1989). An evaluation of several sections on United States (U.S.) Route 10 near Clare, MI, constructed in 1975 to assess the performance of a Jointed Plain Concrete Pavement (JPCP) incorporating different base types, indicated that “the worst-performing sections were the non-doweled JPCP constructed on a dense-graded asphalt treated base (ATB), which retained moisture and accelerated the deterioration of the underside of the concrete slab. The best performing section had a permeable ATB. This pavement section was designed so that no excess moisture was available to accelerate moisture related distresses” (Peshkin, et al. 1989). Another investigation studied roadways as old as 16 years. These pavements constructed with a permeable base suffered premature cracking, spalling, and faulting, but it was concluded that this was the result of trapped water in the subbase, hot weather construction, clogged outlet drains, and high silt and clay content in the subgrade, and not due to the permeable base (Hansen and Van Dam, 1998). The investigators used extensive field and laboratory tests to determine the true

cause of the distresses. This is important to note because often the cause of premature cracking is too quickly ascribed to the permeable base.

2.7.7 Minnesota

A 488-m (1600-ft) jointed reinforced concrete pavement (JRCP), with 8.2-m (27-ft) slabs, was constructed with a permeable base in Minnesota in 1983. After five years, only one slab was cracked, whereas the control section with an undrained dense-graded base had approximately 50% slabs cracked (Mathis, 1989). Another test pavement constructed in 1989 was evaluated after 6 years. Mid-panel cracking was more prevalent in the dense-graded bases sections than in the five sections incorporating an ATPB (Hagen and Cochran, 1996).

2.7.8 New Jersey

In 1988, New Jersey reported that their asphalt concrete pavements constructed with permeable bases were performing as well as their thicker undrained sections. A PCC pavement incorporating a permeable base exhibited less deflection, no faulting nor pumping, and substantially less frost penetration than a pavement without such a base (Mathis, 1989). In 1991, New Jersey continued to report superior performance of drained compared to undrained pavements (Mottola, 1991).

2.7.9 Ohio

The Ohio Department of Transportation (ODOT) reconstructed a length of highway in Erie and Lorain counties in the early 1990s. Six base types, four open-graded and two dense-graded bases, were used in the new test sections. After nine years, the section containing an ATPB continued to outperform all sections, including the two dense-graded sections, but cores taken from the ATPB section show signs of stripping. The “New Jersey” open-graded base and cement treated permeable base were performing poorly and had developed a substantial number of cracks (Sargand and Edwards, 2000). In 2001, ODOT discontinued the use of permeable bases on account of the following concerns: (a) Permeable bases have no effect on the moisture levels; (b) Lack of evidence of pavement performance improvement with permeable bases; (c) Underdrain maintenance and outlet clean-out have not been a high enough priority; and (d) Excessive cost (Lozier, 2001).

2.7.10 Ontario

The Ontario Ministry of Transportation evaluated five different test pavements that included only permeable bases but edge drains or outlets. The pavements ranged in age from approximately 3 to 15 years. Results of the evaluation showed no difference in performance between the permeable base sections and those constructed with a dense-graded base, but the researchers were quick to point out that the performance of the drainable pavement sections would have been better had they been constructed with all the necessary subdrainage components (Hajek, et al., 1992).

2.7.11 Pennsylvania

After seven years of observing the performance of permeable bases, the Pennsylvania Department of Transportation (PennDot) changed its specifications to require the use of permeable bases (Highlands and Hoffman, 1988). At the 1991 *Western States Pavement Subdrainage Conference*, PennDot reported it was still “pleased with its experience” of permeable bases (Burk, 1991).

2.7.12 Virginia

Virginia reported some construction problems with unstabilized permeable bases and no longer considers them. Yet Bowyer (1991) reported that Virginia had success with cement treated permeable bases and is “convinced that the additional [cost] is justified.” To date, all PCC pavements constructed in Virginia utilize either cement or asphalt treated permeable bases and the state is pleased with the results (Maupin, 2004).

2.7.13 Wisconsin

In a five-year performance research project, little difference was found between pavements with permeable or dense graded bases, using the pavement distress index (PDI). Even though little evidence was provided to endorse permeable base, the Wisconsin Department of Transportation (WisDOT) stipulated that “open-graded bases should be used on the Interstate System at all times” (Rutkowski, 1991). Evidence for the usefulness of asphalt stabilized open-graded base courses was collected in a short-term structural and drainage performance study of 12 pavement sections (Rutkowski, et

al., 1998). Researchers recommended Wisconsin open-graded base course #2 over #1, and noted that unsealed transverse joints proved little worse than sealed. Various edge drains performed adequately, but retrofits offered no benefit. Continued evaluation confirmed that the use of cement treated permeable bases was effective (Hall, 1994). A more recent study has concluded that “some of the slab geometry and drainage designs constructed along Wisconsin Route 29 in 1997 and 1999 were found to perform as well as or better than standard designs” (Crovetti, 2006). Currently WisDOT permits the use of permeable bases (WisDOT, 2005).

2.7.14 Nationwide

Under two comprehensive investigations sponsored by FHWA, the Rigid Pavements Performance and Rehabilitation (RPPR) study and the Long Term Pavement Performance (LTPP) program, data were collected and analyzed to evaluate the effects of several pavement design features, including those of base type on the performance of JPCP (Smith, et al., 1998, Jiang, et al., 2001). The LTPP study included test sites from the Special Pavement Studies experiment #2 (SPS-2), and from the General Pavement Studies experiment #3 (GPS-3). The RPPR database pertained to 14 sites in the USA and 1 in Canada. Data analysis demonstrated “the advantage of using permeable bases and treated bases to keep moisture out of the pavement system, and therefore control faulting,” since “sections on permeable bases or lean concrete bases showed the least tendency to undergo faulting for both doweled and undoweled pavements.” Moreover, “for a traffic level of 10 million equivalent single axle loads (ESALs), RPPR sections had

an average joint faulting of 3.1 mm (0.122 in.), 2.4 mm (0.094 in.), and 1.7 mm (0.067 in.) for granular bases, treated bases and permeable bases, respectively.” Additional findings included the following: Treated bases and bases with sufficient drainage showed better performance than untreated bases. Cracking was up to three times higher in pavements with a LCB than in those on ATB or aggregate base. Permeable aggregate bases performed very well and showed the least cracking. Pavements with a drainable base, and/or with an asphalt treated base, had the lowest overall International Roughness Indices (IRI) in the LTPP database.

The RPPR and LTTP databases were also examined under NCHRP Project 1-34, “Performance of Subsurface Pavement Drainage,” completed in 1998, and its three offspring projects (NCHRP 1-34 B, C, and D) (Yu, et al., 1998). Its objectives were “to evaluate (1) the overall effect of sub-surface drainage of surface infiltration water on the performance of asphalt concrete (AC) and PCC pavements; (2) the specific effectiveness of permeable base and associated edge drains, as well as traditional dense-graded bases with and without edge drains; and (3) the specific effectiveness of retrofitted surface drainage on existing pavements.” Findings included the following: Permeable base sections often exhibited better performance with regard to the development of rutting and fatigue cracking compared to dense aggregate base sections, particularly when daylighted or when used with edge drains. Clogged edge drain outlets have a very detrimental effect on pavement performance. A permeable base has a significant effect in reducing joint faulting in non-doweled, jointed concrete pavements, provided it does not become contaminated by fines. The edge drains must also be maintained properly, or they will

clog and their beneficial effect will be lost. When an asphalt-treated permeable base is used with a JPCP, the amount of cracking is very low in comparison with other base types. On the other hand, permeable base layers and edge drains increase the cost of a project significantly; thus, there are always certain tradeoffs involved.

Hall and Correa (2003) reported on the SPS-1 and SPS-2 experiments that evaluated undrained and drained pavements in 18 different states around the country. This study included test pavements in all states listed above and was the largest subsurface drainage study to date encompassing all climatic zones in the United States. Although there was slightly better performance in the drained sections, the difference was not statistically significant and the researchers gave no recommendations regarding the future use of permeable bases.

3 DRAINAGE EVALUATION

3.1 General Information

A convenient example for investigating the performance of permeable bases was recently offered by the United States (U.S.) Route 50 joint sealant experiment near Athens, Ohio, conducted by a University of Cincinnati (UC) research team for the Ohio Department of Transportation (ODOT) between 1997 and 2001. The project is part of a 10.5-km (6.5-mile) stretch of U.S. 50 that was recently reconstructed (Hawkins, et al., 2001). The study pertains to a 3.3-km (2.0-mile) stretch of a four-lane divided highway, approximately 1.3-km (0.8-mile) east of the city of Athens, in Athens County, southeast Ohio. The project lies in the Wet-Freeze climatic zone, where the local mean annual precipitation is 980 mm (38.6 in.). Of this, 533 mm (21 in.) usually accumulates between the months of April and September. During the winter months, the average temperature is 0°C (32°F) and the average daily minimum temperature is -6°C (21°F). The average summer temperature is 22°C (71°F), with an average daily maximum temperature of 29°C (85°F). The mean monthly average temperature is 12°C (53°F). The low average monthly temperature is 0°C (32°F), whereas the high average monthly temperature is 24°C (75°F).

The pavement system consists of a Portland Cement Concrete (PCC) slab, resting on granular base and subbase layers over a predominately silty clay subgrade. The PCC slab is 250-mm (10-in.) thick and is provided with steel wire mesh for control of

temperature and shrinkage cracking (ODOT Item 451). The slab rests on a 100-mm (4-in.) base layer consisting of crushed aggregate that is well-graded and free draining (ODOT Item Special), constructed over a 150-mm (6-in.) crushed aggregate subbase (ODOT Item 304), resting on a silty clay subgrade. In both the eastbound and westbound directions, the highway consists of two 3.7-m (12-ft) wide lanes having tied PCC shoulders. On the inner (i.e., abutting the median) and outer sides of the pavement, the shoulders are 1.2- and 3-m (4- and 10-ft) wide, respectively. Transverse joints, spaced every 6.4 m (21 ft), are fitted with epoxy-coated steel dowels that are 38 mm (1.5 in.) in diameter and 460 mm (18 in.) in length. The dowels are supported on baskets and are placed 305 mm (12 in.) on center, starting at 150-mm (6-in.) from the shoulder joint. The longitudinal center line and shoulder joints are tied with 16-mm (0.625-in.) diameter, 760-mm (30 in.) long deformed steel bars spaced every 760 mm (30 in.). The test pavement was given the functional classification of rural principal arterial, and it was designed to carry approximately 11 million equivalent single axle loads (ESALs). The level of reliability selected was 85.0% (Ioannides, et al., 2001).

Construction of the test pavement occurred in two phases. Construction of the eastbound lanes began in the Summer of 1997 and these were opened to traffic in Spring of 1998. During this first construction phase, both directions of traffic were served by the existing asphalt concrete (AC) pavement. Subsequently, traffic was diverted from the existing highway to the newly constructed eastbound lanes. This allowed the second phase of construction to begin in the Summer of 1998. The westbound lanes were opened to traffic in May 1999.

Heavy rainfall caused localized flooding along stretches of the test pavement in Spring 2000 (Ioannides, et al., 2004). Several of the longitudinal ditches were flooded and the ground was saturated throughout the area. Large areas of ponded water were noted in the drainage swales alongside both directions of the highway, with water levels appearing to be almost at the elevation of the pavement subgrade surface.

3.2 Site Evaluations

It was postulated that perhaps poor drainage was responsible for some of the premature cracking observed at the site, prompting an investigation into the drainage features of the pavement. To begin with, the outlets were inspected in Fall 2001 (Ioannides, et al., 2001). These are the locations where water is transferred from the pavement base to the surface drainage channels. At the test site, outlets are unmarked and made of pre-cast concrete that is designed to protect the tip of the collector pipe and reduce erosion. Screens are attached to the outlets to prevent small rodents from creating nests in the collector pipes.

3.2.1 Outlet Locations

The locations of the outlets are indicated in the project construction drawings (ODOT, 1995) and are listed in Table 3.1. Only the outlets within the area of the sealant experiment were inspected. The Table also records if the outlet was actually found, if the rodent screen was in place, the amount of silt and vegetation present, and the presence of

standing or flowing water.

Only 19 of the 26 outlets listed in the project construction drawings within the project span were located. Of the outlets that were found, many were engulfed by tall vegetation that had to be cleared before examination (Figure 3.1). Although portions of the roadside were mowed, the area where the outlets are located had not been (Figure 3.2), and this probably explains why some of the outlets were not located. Apparently such regions are intentionally left un-mowed for environmental conservation reasons (Bob McQuiston, 2001: personal communication). Other locations did not have such vegetation, yet a number of outlets were not found there either. These were probably never constructed in accordance with the original ODOT (1995) plans, and no post-construction shop drawings were available. This experience is not atypical. Baldwin and Long (1987) conducted a similar evaluation in the mid 1980s, inspecting drain outlets once a year for three years, and never found more than 60% of the outlets. Compared to their efforts, the number of outlets discovered at the Athens project site (73%) may be considered remarkable.

3.2.2 Collector Pipes

The transverse collector pipes that run perpendicular to the roadway and allow an outlet for the draining base were also inspected. It is impossible to view deep within the collector pipes without the aid of expensive equipment (e.g., borehole camera) and because an expenditure of this magnitude could not be justified on the current project such devices were not used. Consequently, collector pipes could be viewed only near the

outlet with the help of a flashlight or using the infrared feature on a video recorder (Figure 3.3). Most of the outlets found had large amounts of silt, moss, and weeds in them and at times this combination was several inches thick (Figure 3.4). One outlet even had a weed growing out of the rodent screen that measured several meters (feet) high (Figure 3.5). All outlets were checked for standing or flowing water, which is a telltale sign that the collector pipes are not functioning properly. Some drains were found completely dry, which may indicate that the pipe is either broken or clogged since water cannot reach the outlet. The surrounding area was subject to frequent rainy weather prior to the evaluation and the dry outlets are not likely to be the result of a dry base.

After observing several of the outlets, it was noticed that those with large gradients were relatively free of silt and debris and water was freely flowing from the collector pipe. ODOT (1995) specifications call for a 1% pipe slope, but it is difficult to ascertain precisely if the drains are at the required gradient. Nonetheless, there is an obvious correlation between steeper slopes and outlet functionality.

3.2.3 Rodent Screens

Rodent screens are typically placed at the end of the collector pipe to prevent small rodents from entering the pipe and building nests. None of the outlets in the eastbound lanes have rodent screens, whereas all but one of the outlets in the westbound lanes do have them. Although screens are required by specifications, their absence may actually be beneficial. Moss, weeds, and eventually silt are present on most screens and can transform them into small dams (Figure 3.6). When the screens were temporarily

removed, water gushed out and the collector pipe was free to drain. It is apparent that the rodent screens are not even capable of serving their intended purpose, as several of them had been bent, creating a gap large enough for small rodents to fit through (Figure 3.7). It is unclear why these screens had been bent, but two hypotheses emerge: (a) The screens had been bent on purpose during construction, possibly to provide a snugger fit with the concrete outlets, even though this is not a technique endorsed by ODOT; and (b) they had been bent accidentally during construction or subsequent periodic mowing operations. It is evident that the rodent screens serve no beneficial purpose and actually are detrimental to the drainage process.

3.2.4 Outlets

The pre-cast reinforced concrete outlets are generally in good condition with only few distresses observed. One exception is outlet No. 209, which appears to have been recycled from another roadway. This particular outlet is noticeably older as evidenced by the discoloration and deterioration of the concrete. Furthermore, it had been improperly placed and did not provide adequate protection to the collector pipe. Consequently, the pipe had been crushed and its lip formed a "V" at the tip, impeding water flow. Another outlet (No. 143) had slid down the hillside and left the collector pipe completely exposed (Figure 3.8). The pipe seemed to be in good condition but was not inspected too closely due to the presence of a snake, which was sunbathing on the concrete outlet (Figure 3.9). The UC research team felt it was better to leave the pipe uninspected rather than disturb the reptile!

3.2.5 Markers

There were no outlet markers of any kind found at the Athens project site. An outlet marker is a posted sign that clearly shows the location of an outlet. It should be on a protective post tall enough to be seen over the vegetation growth. The researchers had difficulty finding the outlets due to the extensive overgrowth. Some of the outlets are not located as specified in the ODOT specifications and plans, but are often found within the general location. Outlet markers would be most beneficial in locating such outlets.

3.2.6 Maintenance

It was apparent that little or no maintenance had been performed on the drainage outlets. Most outlets were clogged with silt and debris and upon their removal water poured out of the collector pipe. Such observations reaffirm the need for regular maintenance of the outlets to prevent the base layer from prolonged periods of saturation. The underdrains should be periodically cleaned or flushed to prevent the accumulation of silt and debris. This process can be aided by the placement of clean-out boxes (Moulton, 1980), but no such devices are located on the U.S. 50 project site. Therefore, flushing may be more difficult and less effective.

A similar inspection process was conducted by Hassan, et al. (1996) with eerily similar results. A lack of maintenance at those researchers' site led to the demise of the pavement's subsurface drainage system. The researchers found outlet pipes exposed, crushed, or punctured. No outlet markers were present on site making the identification of some outlets difficult or impossible due to heavy vegetation growth. Rodent screens

were absent or did not cover the entire pipe outlet. Vegetation growth restricted or completely blocked the flow of water within the outlet pipe. The researchers attributed the deterioration of the jointed reinforced concrete pavement (JRCP) within 3 years to the lack of maintenance.

Table 3.2 lists recommended performance criteria as stated by Mallela, et al. (2001). It is clear that the performance of the subsurface drainage at the U. S. 50 test pavement fails to meet these criteria, an obvious consequence of the lack of maintenance at the project site.

3.3 Computer Software Calculations

Design calculations for the drainage structures at the Athens project were performed upon learning that no such calculations had been conducted prior to construction (Aric A. Morse, 2001: personal communication). Calculations utilized the software program “Drainage Requirements in Pavements” (*DRIP 2.0*) and are described herein. Background information for the equations and the theory behind them is provided in detail by Wyatt, et al. (1998).

Computer program *DRIP 2.0* was developed for the Federal Highway Administration (FHWA) to meet the need of pavement engineers for a concise and user-friendly tool to aid in the design of subsurface drainage systems in pavements. *DRIP 2.0* replicates the efforts of the *Drainable Pavement Systems: Participant’s Notebook* (FHWA, 1992), which provides guidance regarding subsurface drainage design. The

program is organized into six input modules: Roadway Geometry; Inflow; Permeable Base; Edge drain; Separator Layer; and Gradation Analysis. It is designed to provide the user with three essential outputs: permeable base thickness; edge drain outlet spacing; and separator layer-to-subgrade compatibility. The following paragraphs describe the input requirements of these modules, including values used for the U.S. 50 evaluation.

3.3.1 Roadway Geometry

The Roadway Geometry module allows the user to input the geometry of the pavement (width, cross slope, etc.) and to determine the longest flow path (L_R) and the resultant slope of the draining base (S_R). These outputs are critical in determining the proper draining base thickness. The user has the option of two roadway configurations, Geometry A (crowned) and Geometry B (super-elevated). From ODOT (1995), it is apparent that Geometry B is appropriate for the site-specific calculation. The input parameter b represents the total width of the pavement (7.3 m or 24 ft) and c represents the horizontal distance from the edge of the pavement to the edge drains (0 m or ft). The required permeable base width (W) is then equal to 7.3 m (24 ft).

The longest flow path length (L_R) is a function of the permeable base width (W), the cross slope (S_x), and the longitudinal gradient (S) of the roadway. The latter was varied from 0 to 10% at increments of 1% over the length of the project. As a result, several values of S_R and L_R were calculated, leading to multiple thickness values for the draining base. No calculations had been performed to establish the required base

thickness during the design of the pavement, but a standard open-graded thickness of 100 mm (4 in.) was provided.

3.3.2 Inflow

The Inflow module in *DRIP 2.0* calculates the volume of water infiltrating the pavement, according to two distinct methods. The first of these is more empirical and is based on the assumption that a fixed percentage (or ratio) of rainwater falling onto the pavement surface will infiltrate it. The FHWA (1992) suggest considering the 1-hour duration, 2-year frequency rainfall event for this calculation, but *DRIP 2.0* can also accommodate the less severe 1-hour duration, 1-year frequency rainfall event; both were utilized for this calculation. Values of 26 and 33 mm/hr (1.02 and 1.3 in./hr) were selected for the 1-year and 2-year rainfall events, respectively, based on the geographic location of the project site (http://hdsc.nws.noaa.gov/hdsc/pfds/orb/oh_pfds.html). Infiltration ratios suggested by Cedergren (1974) range from 0.50 to 0.67 for PCC pavements and 0.33 to 0.50 for asphalt pavements; a conservative value of 0.67 was chosen for the U.S. 50 project site. This results in calculated pavement infiltration rates, q_i , of 0.418 (1.37) and 0.531 $\text{m}^3/\text{d}/\text{m}^2$ (1.74 $\text{ft}^3/\text{d}/\text{ft}^2$) for the 1-year and 2-year rainfall events, respectively.

The second method in *DRIP 2.0* is more mechanistic, and considers the number and length of cracks or joints, as well as their ability to conduct water. The latter is quantified by the crack infiltration rate, I_c , for which a conservative value of 0.223 $\text{m}^3/\text{d}/\text{m}$ (2.4 $\text{ft}^3/\text{d}/\text{ft}$) is suggested by the FHWA (1992). Adopting this value, gives rise to a q_i rate of 0.079 $\text{m}^3/\text{d}/\text{m}^2$ (0.26 $\text{ft}^3/\text{d}/\text{ft}^2$).

3.3.3 Permeable Base

Outputs from the Geometry and Inflow modules are input into the Permeable Base module to determine the minimum base thickness ($H_{\min}$) required. Inputs into this module are S_R and L_R from the Roadway Geometry module, q_i from the Inflow module, and the coefficient of permeability (k) of the base. The U.S. 50 project contains two drainable base materials, “Iowa” (IA) and “New Jersey” (NJ). These gradation types have estimated hydraulic conductivities of 153 and 610 m/day (500 ft/day and 2000 ft/day), respectively (Mathis, 1989).

Referring to Figure 3.10, permeable base thickness requirements for the IA gradation quickly become excessive as the longitudinal gradient increases. The trend is less significant in the NJ gradation, which has a higher coefficient of permeability. Intuitively, one might think that the thickness requirement would decrease as the hydraulic gradient increased. Drainage requirements of a permeable base, however, depend on the drainage length, L_R . As the longitudinal gradient increases, water tends to flow longitudinally rather than laterally, thus increasing the drainage length and time for water to escape to the edge drains. This increase in drainage length is more significant than an increase in hydraulic gradient.

One way to decrease the drainage length is to increase the cross slope. Calculations performed by Croveti and Dempsey (1993) show permeability requirements nearly cut in half at higher longitudinal gradations by increasing the cross slope from 1% to 2%. The U.S. 50 pavement incorporates a 1.5% cross slope, and Figure 3.11 shows the results of increasing the cross slope from 1.5% to 2.0%. The increase in cross slope

results in a decrease in base thickness of nearly 300 mm (1 ft) for the IA gradations at higher longitudinal gradients. This exercise clearly shows the sensitivity of a less permeable base and the importance of a base with significant permeability and cross slope.

Calculations show that only the NJ gradation results in a permeable base of sufficient thickness at the project site provided the crack infiltration method is used. Clearly the IA gradation (having a permeability of only 153 m (500 ft) per day) is inadequate as a permeable base and should not be considered for future use. Recall that other states, such as Illinois, require permeability between 305 and 610 m (1,000 and 3,000 ft) per day (Croveti and Dempsey, 1993). It is uncertain and beyond the scope of this project to determine whether the high degree of premature cracking at the U.S. 50 project site is attributable to the lack of design of the permeable base, but the project offers an example of how drainage calculations are often ignored or improperly used.

3.3.4 Edge Drain

The Edge Drain module calculates the maximum undrained length of the edge drains (outlet spacing) by three different methods: pavement infiltration flow rate, peak flow for the permeable base, and average flow rate during the time-to-drain the permeable base. This module is designed to provide the drainage engineer with maximum spacing requirements for the drainage outlets.

Input variables for the pavement infiltration flow rate method include: longitudinal gradient (S), edge drain diameter (D), and edge drain roughness coefficient

(n), as well as pavement infiltration flow rate (q_i) and permeable base width (W). Given that a 100-mm (4-in.) diameter corrugated pipe was used for the U.S. 50 road project (ODOT, 1995), values of 100 mm (4.0 in.) and 0.024 are used for D and n , respectively. A constant permeable base width of 7.3 m (24 ft) is used with the three different pavement infiltration flow rate values noted earlier ($q_i = 0.418, 0.531, \text{ and } 0.079 \text{ m}^3/\text{d}/\text{m}^2$ or $1.37, 1.74, \text{ and } 0.26 \text{ ft}^3/\text{d}/\text{ft}^2$).

The second method to calculate outlet spacing utilizes the peak flow for the permeable base. Input values are again longitudinal gradient (S), edge drain diameter (D), and edge drain roughness coefficient (n), but also include permeable base thickness (H), hydraulic conductivity (k), and cross slope (S_x). For the test site, $H = 100 \text{ mm}$ (4.0 in.) and $k = 153 \text{ and } 610 \text{ m/day}$ (500 and 2000 ft/day) for the IA and NJ gradations, respectively.

The final method is based on the time for drainage to be reached, which is a function of the effective porosity (n_e) of the subgrade. Because of the existence of multiple subgrade soils at the project site, and the lack of laboratory testing, this method was not evaluated. The results of the first two methods provide ample data to come to a firm conclusion concerning outlet spacing.

Calculations suggest that as the longitudinal slope (S) approaches zero so does the edge drain spacing, since there is no hydraulic gradient forcing the flow of water (Figure 3.12). Conversely, as the longitudinal gradient increases the edge drain spacing increases, since the higher hydraulic gradient is sufficient to overcome larger undrained distances. Obviously tight outlet spacing is impractical and costly so mild longitudinal slopes should

be avoided when possible. It is found that the maximum undrained length values rise quickly above 153 m (500 ft) at just 1% longitudinal slope. Although edge drain spacing at the U.S. 50 project site is variable, outlet intervals are generally less than 153 m (500 ft) apart, suggesting satisfactory design.

3.3.5 Separator Layer

The test pavement was constructed without a separator layer between the drainable base and the subgrade, but the 150-mm (6-in.) dense-graded subbase effectively serves as such (Aric A. Morse, 2001: personal communication). At the U.S. 50 project site, twelve different subgrade soil types were reported (ODOT, 1995) and each was investigated for gradation compatibility with the subbase as separator layer. *DRIP 2.0* offers a convenient gradation analysis module for this purpose that calculates C_u , D_{15} , D_{50} , and D_{85} values. The results of the gradation analyses are listed in Table 3.3. Only a third of the subgrade soils found at the Athens project site are compatible with the separator layer, which is not fine enough to filter the fine particles of the subgrade. The compatible soil types are all gravelly soils. The open-graded base, therefore, is found to be susceptible to the infiltration of fines, which may lead to clogging and reduction of its drainage capabilities

3.4 Drainage Conclusions and Recommendations

The inclusion of open-graded bases in the design of PCC pavements seeks to ensure adequate drainage, but such layers must be properly maintained. If silt, moss, and other debris are allowed to accumulate in the collector pipes, water will not be able to escape and the base will become saturated. To keep the pipes draining freely, a routine maintenance program must be implemented. Maintenance should consist of cleaning the outlets of any vegetation overgrowth that may hamper future efforts to locate the outlet. Once the outlet is found, a marker should be installed that clearly identifies its location so it may be found easily in the future. All silt, moss, and debris should be removed at the outlet and from the rodent screen. Flushing is suggested by Moulton (1980) but without the aid of clean-out boxes it may be rather difficult to perform. Rodent screens should be inspected for damage and a redesign may be necessary to ensure a snug fit without any gaps. The gradient of the transverse collector pipes should be increased to produce a higher exit velocity so that silt and debris cannot gather in the pipe. Non-perforated metal or smooth, rigid pipes may resist clogging more effectively.

It is apparent from the drainage evaluation and computer software analyses that the subsurface drainage features at the U.S. 50 project site lack proper design and maintenance. The draining base constructed under the slab does not have sufficient permeability or thickness. It is determined that the base thickness could have easily been more than twice that provided. This is especially true for the westbound lanes, which contain the less permeable IA gradation. Analytical results suggest base thickness values

for this gradation to be in the range of 0.23 to 1.14 m (0.75 to 3.75 ft), which is quite wide and unrealistic. Consequently, the engineer is left without meaningful guidance with regard to base thickness selection, since even averaging such results would be arbitrary.

Evidently, gradations with permeabilities below 2000 ft/day are unsuitable in permeable bases. Moreover, it appears that a cross slope of 2% or more can be beneficial, as can positive measures to ensure base-subgrade compatibility, including the use of a separation layer or geotextile. It is recommended that the more mechanistic crack infiltration method implemented in *DRIP 2.0* be preferred.

Table 3.1 Location and condition of underdrains

	Underdrain No.	Station	Offset	Found	Screen	Silt	Vegetation	Water
EASTBOUND	84	155+00	95' RT	YES	NO	LOW	HIGH	STANDING
	92	152+68	122' RT	YES	NO	LOW	HIGH	NONE
	109	170+00	98' RT	YES	NO	HIGH	HIGH	NONE
	115	174+50	103' RT	YES	NO	HIGH	HIGH	STANDING
	121	178+97	104' RT	YES	NO	LOW	HIGH	-
	136	184+00	106' RT	YES	NO	NONE	HIGH	FLOWING
	143	199+00	84' RT	YES	NO	N/A	HIGH	N/A
	149	202+97	81' RT	YES	-	HIGH	HIGH	NONE
	155	206+74	72' RT	YES	NO	HIGH	HIGH	STANDING
	166	213+00	62' RT	YES	NO	LOW	HIGH	FLOWING
	171	218+00	70' RT	YES	NO	HIGH	-	NONE
	209	257+00	81' RT	YES	NO	HIGH	HIGH	STANDING
	226	272+75	86' RT	NO	-	-	-	-
	232	279+97	100' RT	NO	-	-	-	-
	238	280+03	100' RT	NO	-	-	-	-
	244	291+00	93' RT	NO	-	-	-	-
WESTBOUND	82	148+50	95' LT	YES	YES	HIGH	HIGH	STANDING
	89	155+00	93' LT	YES	YES	NONE	LOW	FLOWING
	94	152+00	105' LT	YES	NO	NONE	HIGH	STANDING
	114	170+00	90' LT	YES	YES	NONE	LOW	NONE
	120	174+50	95' LT	NO	-	-	-	-
	132	184+50	95' LT	YES	YES	NONE	LOW	FLOWING
	221	261+00	82' LT	NO	-	-	-	-
	237	276+00	98' LT	YES	YES	NONE	HIGH	STANDING
	243	280+00	98' LT	NO	-	-	-	-
	249	291+00	81' LT	YES	YES	NONE	LOW	FLOWING

Table 3.2 Recommended Performance Criteria (from Mallela, et al., 2001)

Asset Item	Performance Goal	Performance Target (%)
Edgedrains/Underdrains	<ul style="list-style-type: none">• Functional and not clogged• No crushed or ruptured pipes• Free of debris and animal nests• Well-connected to outlets	90
Outlets/Cross-pipes/Laterals	<ul style="list-style-type: none">• Clearly visible and above the design flow line in ditch• 90 percent or greater area open• Not crushed or ruptured• Well connected to edge drains• Laid out a 3% slopes	100
Headwalls	<ul style="list-style-type: none">• Visible, intact, and provide protection to outlet• Minimal erosion of surrounding soil	80
Rodent screens	<ul style="list-style-type: none">• Present and functional	75
Outlet markers	<ul style="list-style-type: none">• Present and visible	100

Table 3.3 Results of gradation analysis and separator layer compatibility test

Subgrade Description	Percent Passing (%)			Particle Size (in.)		Compatibility
	#10	#40	#200	D₅₀	D₈₅	
Gravel	39	21	9	0.1329	0.4507	Yes
Gravel and Sand	73	35	13	0.034	0.1023	Yes
Fine Sand	96	75	7	0.0102	0.0385	Yes
Coarse and Fine Sand	94	74	22	0.0088	0.0427	Yes
Gravel with Sand and Silt	68	52	31	0.0148	0.1884	No
Gravel with Sand, Silt, and Clay	61	45	32	0.0298	0.2525	No
Sandy Silt	95	91	62	0.0018	0.0128	No
Silt	98	97	86	0.0008	0.0028	No
Silt and Clay	90	84	74	0.0012	0.0232	No
Silty Clay	88	81	71	0.0013	0.044	No
Elastic Clay	98	96	87	0.0008	0.0028	No
Clay	87	82	76	0.0011	0.0458	No

Figure 3.1 Tall vegetation made finding the outlets difficult

Figure 3.2 Mowed and unmowed areas

Figure 3.3 View of inside of collector pipe using the infrared device

Figure 3.4 Combination of silt, moss and weeds that has collected in the outlet

Figure 3.5 A large weed growing out of one of the outlets

Figure 3.6 Moss and silt that has gathered on the rodent screen

Figure 3.7 The rodent screen has been bent back, creating a gap for small rodents

Figure 3.8 A concrete outlet that has slipped down the hillside, exposing the drain

Figure 3.9 A snake sunbathing on the concrete outlet

Figure 3.10 *DRIP 2.0* permeable base thickness calculations with 1.5% cross slope

Note: 1 ft = 0.305 m

Figure 3.11 DRIP 2.0 permeable base thickness calculations with 2.0% cross slope

Note: 1 ft = 0.305 m

Figure 3.12 DRIP 2.0 edge drain spacing calculations

Note: 1 ft = 0.305 m

4 SEALANT AND PAVEMENT PERFORMANCE

4.1 Introduction

This Chapter presents a summary of the performance of a variety of joint sealants at the United States (U.S.) Route 50 test site in Athens, Ohio, approximately after three years in service. In addition, profile smoothness and structural distress data are examined in order to determine whether any correlation exists between sealant performance and pavement slab condition. A complete report has been submitted to the Ohio Department of Transportation (ODOT) under different cover that included the design and construction of the U.S. 50 test pavement, experimental design for the sealant investigation, monitoring activities, and the sealant and pavement performance (Ioannides and Minkarah, 2002). The reader is also directed to two prior works published in the technical literature (Hawkins, et al., 2001; Ioannides, et al., 2001), as well as two previous interim reports submitted to ODOT by the research team (Hawkins, 1999; Sander, 2002).

The provision of discontinuities in Portland cement concrete (PCC) pavements has been a major construction concern since the need for contraction joints was first recognized in the early 1900s. Joints at regularly spaced intervals were introduced in concrete pavements to control cracking attributed to the restraint of free movement of slabs during curing. Because such discontinuities were invariably planes of weakness, distresses in concrete highways were often noted to initiate and rapidly propagate at or near the slab edges. This prompted many engineers, scientists and other researchers to

embark on numerous studies in an effort to develop the most efficient and durable joints. A significant focus of such investigations has been to protect the joint and the area surrounding it from premature distress by using joint sealants.

After decades of research, the conventional wisdom has evolved that sealing joints with impervious materials helps prevent or minimize the infiltration of moisture and of incompressible debris into the pavement system, thereby leading to improved pavement performance over a longer period of time. Free water entering a joint can accumulate beneath the pavement slab and result in such distresses as loss of base or subgrade support, faulting, transverse cracking and corner breaks. Additionally, as joints become filled with incompressible materials, slab thermal expansion is restrained, resulting in significant spalling at the joint lips.

It is tempting to postulate that the success or failure of a jointed PCC pavement system may be attributed—at least in part—to the success or failure of the joint sealants. This, however, may not hold true in many cases. Concrete pavement performance is a complex phenomenon that is dependent on many parameters and not just on properly sealed joints. If the pavement is designed with sufficient excess capacity to withstand the applied loading and environmental stressors, then perhaps effectively sealed joints are not necessary in achieving satisfactory performance levels. A pavement that is constructed with features that positively affect performance, e.g., load transfer, subsurface drainage, thickened edges, etc., may not require sealed joints in order to maintain high levels of serviceability. Conversely, distresses in the form of faulting, cracking or spalling may occur in a pavement regardless of sealant presence or condition, simply as a consequence

of other design inefficiencies. Therefore, any experiment intended primarily to study joint sealants cannot afford to disregard everything else that contributes to pavement performance.

4.2 Sealant Performance

The experimental design, performance evaluation methodology and data analysis methodology for this project conform to guidelines established by previous studies, particularly those under the Strategic Highway Research Program (SHRP) Specific Pavement Studies (SPS)-4 sealing effectiveness initiative (Smith, et al. 1999). Ten different sealant compounds were used for sealing transverse joints in the test pavement. Four of these sealant materials are silicone, two hot-applied (hot-pour) and four preformed compression seals, as follows: Crafcro 903-SL, Dow 890-SL, Crafcro 902, and Dow 888; Crafcro 221 and Crafcro 444; and Delastic V-687, Watson Bowman WB-687 and 812, and Techstar W-050. Four test sections were intentionally left unsealed to evaluate the effects of unsealed joints on pavement performance. In this experiment, six joint configurations or designs (numbered 1 through 6) were used. Only configurations 1, 3 and 5 received a secondary cut, and backer rod was placed in designs 1, 3 and 4 only. Configurations 2 and 6 were used in unsealed test sections, whereas designs 1, 3 and 4 were used for liquid sealants. All transverse joints requiring the use of a compression seal had joint configuration 5. Nominal joint width varies by joint configuration, as follows: 1 and 5: 9.53 mm (3/8 in.); 2 and 4: 3.18 mm (1/8 in.); 3 and 6: 6.35 mm (1/4

in.). By combining the various sealant materials and joint configurations, a total of fifteen different treatments were formed, e.g., TechStar W-050 (5).

Evaluations to date include the following: (a) two profile surveys and two visual inspections of the eastbound lanes, as well as a single profile survey and visual inspection of the westbound lanes, conducted prior to Fall 1999 and reported by Hawkins, et al. (2001); (b) two profile surveys and two visual inspections of the both sides of the test pavement conducted in Fall 1999 and Spring 2000 and discussed by Ioannides, et al. (2001), who also presented an initial data set pertaining to the development of structural distresses in the pavement slab, notably transverse and corner cracking; and, (c) three profile surveys and three visual inspections in accordance to the same evaluation plan first implemented in Fall 1999, conducted in Fall 2000, Spring 2001 and Fall 2001. The latter three evaluations are examined in detail in the Final Report for the project (Ioannides and Minkarah, 2002) and Ioannides, et al. (2004), from which information in this Chapter is excerpted.

4.2.1 Performance of Eastbound Lane Seals as of EBOC01

The compression seals are the superior sealant material in the eastbound lanes, averaging 69% effectiveness, even when including the TechStar W-050 (5) section, which is only 19% effective. If this section is excluded, the compression seals' average becomes 94%, which is the effectiveness value for both Watson Bowman 687 (5) and Delastic V-687 (5). The two hot-applied sealants differ quite dramatically from one another. Crafc0 221 (1) has the third highest effectiveness value (79%), yet Crafc0 444

(1) has the lowest value (9%). These two sections average 44% effectiveness, the lowest among the three sealant types. The silicone sealants average 46% effectiveness, which is only slightly better than the hot-applied. The two self-leveling sealants with the No. 1 joint configuration are the best performing silicone sections to date. Dow 890-SL (1) and Crafc0 903-SL (1) have effectiveness values of 71% and 58%, respectively. Only one other silicone section is above 50%, namely Dow 890-SL (3), which is 57% effective. The remaining five sections are below 50% effectiveness, including Crafc0 903-SL (4), which is only 12% effective.

4.2.2 Performance of Westbound Lane Seals as of WBOC01

Partially due to their ‘younger’ age, the westbound sealants are performing much better than those in the eastbound lanes. The westbound compression seals, with the exception of TechStar W-050 (5), continue to perform exceptionally well. Watson Bowman 812 (5) and Delastic V-687 (5) are 98% and 97% effective, respectively. The average of the compression seals (66%) is depressed due to the ineffectiveness of the TechStar W-050 (5) section. Excluding this section yields an average of 98%, which is best amongst all the sealants. The difference in effectiveness between the two superior compression seal sections and TechStar W-050 (5) could not be greater. The latter is only 4% effective, and its ability to keep any water out of the joint is extremely questionable. The difference between the two hot-applied sections is pronounced: Crafc0 444 (1) is 93% effective, yet Crafc0 221 (1) is only 43% effective making the difference 50%. The silicone sealants average 85% effectiveness, which is the best for the

westbound lanes (when TechStar W-050 is included among the compression seals). All silicone sealants with the No. 1 configuration, with the exception of Crafc0 903-SL (1b), are in very good condition ($> 90\%$). The only No. 3 configured section, Dow-SL 890 (3), has the highest effectiveness value (99%) out of all the westbound sections. The two No. 4 configured silicone sections, Crafc0 903-SL (4) and Dow 890-SL (4), have effectiveness values of 85% and 44%, respectively. It is apparent that the narrow No. 4 joint configuration tends to result in poorer effectiveness values.

It is interesting to examine the performance of the sealants over the entire duration of the project, and to compare the observations made in each of the two directions of the highway, accounting for the difference in age between the eastbound and westbound lanes. The following is a summary of this information.

4.2.3 Performance of Eastbound Lane Seals Over Entire Project

The effectiveness values for all sections and surveys over the course of the project are shown in Figure 4.1, which clearly portrays the performance trend over time for these sealants. The nearly identical excellent performance of the two compression seal sections, Watson Bowman 687 (5) and Delastic V-687 (5), is most noteworthy. The effectiveness values of these two seals never differ by more than 1% from one another. Their long-term performance looks promising, whereas the third compression seal, TechStar W-050 (5), seems doomed for a quick ultimate failure; the performance of the latter has fallen precipitously well below that of the other two compression seals. It is apparent that from the beginning Crafc0 444 (1) began deteriorating at a faster rate than

the other hot-applied section, Crafc0 221 (1). The former appears to be at a terminal effectiveness level since it does not have much more effectiveness to lose, whereas the latter is maintaining its residual effectiveness. Generally, the silicone sealants have steadily declined in effectiveness since their installation. No section is currently above 75%, and five of the eight are below 50%, including Crafc0 903-SL (4), which is only 12% effective. The sealants in these sections do not show much promise for the long-term.

It is noted that nine of the thirteen sections exhibit increases in effectiveness. Such observations are a great concern to the University of Cincinnati research team. The larger increases are confined to the silicone sections, which show how difficult it is to evaluate this type of sealant. The very large increases (about 50%) are found in the sections with No. 4 joint configuration. These joints are the narrowest of the test joints, with a nominal width of 3 ± 2 mm ($1/8 \pm 1/16$ in.), a feature that makes it difficult to determine objectively and with confidence whether a water-tight bond exists between the joint walls and the sealant. It should be noted that for the sake of objectivity, the University of Cincinnati research team does not refer to previous data sets prior to collecting a new one. A possible explanation for the apparent improvements in effectiveness may be found through a comparison of notes taken during two consecutive surveys, by two different inspectors. This reveals that the scale and degree of detail of the observations, as well as the subjective opinion of the evaluator, may play a more significant role than previously realized. Moreover, changes in the joint width opening in

response to temperature variations shortly before each evaluation may be additional contributors, especially in the case of narrow joints.

4.2.4 Performance of Westbound Lane Seals Over Entire Project

The performance of all the westbound sections over their entire life to date is shown in Figure 4.2. This indicates that two of the compression seals, Watson Bowman 812 (5) and Delastic V-687 (5), have maintained nearly all of their original effectiveness, which promises excellent performance in the future, as well. In contrast, the third compression seal, TechStar W-050 (5), may have at one time been 100% effective, but deteriorated quickly soon after its installation. It is clear that this section has been steadily declining in effectiveness over the past three years. The hot-applied sealants were not installed until April 1999, whereas all other seals in the westbound lanes had been installed in December 1998. Unlike the eastbound lanes, where Crafc0 444 (1) began deteriorating very rapidly and dramatically, the corresponding westbound section has lost very little effectiveness and it has generally maintained effectiveness values above 90% for its lifetime to date. Crafc0 221 (1) deteriorated rapidly early on, but more recently it has maintained a steady effectiveness value. Most of the silicone sealants have maintained much of their original effectiveness throughout their lifetime. Four sections, Dow 890-SL (3), Dow 888 (1b), Dow 890-SL (1), and Crafc0 903-SL (1a), have never dropped below 95% effectiveness. Dow 888 (1a) recently dropped to 91%, but it had been above 95% in all previous surveys. Crafc0 903-SL (4) and Crafc0 903-SL (1b) had deteriorated to 89% and 77%, respectively, during WBMRO0, but they have essentially

maintained those values since then. The two identical Crafc0 903-SL (1) sections are performing quite differently. Crafc0 903-SL (1a) is outperforming its twin by approximately 20% throughout the time span considered. The Dow 890-SL (4) section is very hard to survey due to the very narrow joint width, which makes it difficult to determine adhesion failure and may be responsible for fluctuations in recorded effectiveness values.

It is noted that apparent improvements in sealant effectiveness between consecutive evaluations are more rare for the westbound than the eastbound sections, and generally occur at the time when there was a change in the research team personnel.

4.2.5 Comparison of Performance of Eastbound and Westbound Lane Seals Over Entire Project

It is impossible to make a direct comparison between the eastbound and westbound sealants during any single survey. Consequently, the data from each survey must be expressed in terms of time elapsed since the highway was opened to traffic in each of the two directions, so that sealants of a similar age can be compared. Figure 4.3 compares the results of the westbound survey of October 2001 (WBOC01) with those from the eastbound evaluation conducted one year earlier (EBOC00). At the respective times of these assessments, all sections were approximately three years old. Even when compared to the eastbound sealants at the same age, the westbound lane seals are observed to be performing much better than those in the eastbound lanes. Only the compression seals are performing similarly to their counterparts in the opposite lane

direction. The westbound silicone sealants are outperforming the eastbound sealants at the same age by 47%. The same comparison for the hot-applied sealants yields an average difference of 29% in favor of the westbound sealants. The U.S. 50 sealant experiment includes many sealant materials and joint configurations not normally utilized in Ohio. It is reasonable to expect that the sealant installation crew was less familiar with some treatments than with others. Because the westbound sealants were installed a year after those in the eastbound lanes, the crew may have benefited from their first year experience, making the installation process more effective in the second. The similarity in the behavior of the eastbound and westbound lane compression seals that are commonly used in Ohio, corroborates this assertion.

The Watson Bowman and Delastic seals in both the eastbound and westbound lanes are performing extremely well. The eastbound TechStar section is outperforming its westbound counterpart, but effectiveness trends are rather poor. In both directions, this material exhibits less than 20% effectiveness, and continues to deteriorate. It is believed that these sealants are not designed to adhere to PCC since they are manufactured specifically for bridge decks. A large discrepancy between the two Crafc 444 (1) sections is evident. The eastbound section never performed as well as the westbound, which hints at possible deficiencies in the installation of the former. It is possible that the construction crew gained experience with the installation of the eastbound section, and used this effectively during the installation of the westbound. Moreover, it is possible that delaying the westbound installation until the following Spring was very beneficial. The Crafc 221 (1) sections, however, do not support these

postulates. Just the opposite is observed in these sections, albeit to a much lesser degree: the eastbound are outperforming the westbound. The effectiveness difference between the two lane directions is about 25% for the Crafc0 221 (1), whereas for the Crafc0 444 (1) sections this difference is about 80%. All of the westbound silicone sealant sections are outperforming their eastbound counterparts by a large margin over their lifespan to date. Every section currently has at least 25% more effectiveness than its counterpart. It is apparent that all eastbound sections never performed as well as their westbound counterparts. The westbound Dow 888 (1) sections have never dropped below 90% effectiveness, whereas the eastbound deteriorated drastically very early on and are currently about 50% below the westbound lanes. This suggests that poor workmanship may be responsible for the dismal performance of the eastbound silicone sealants. Westbound Dow 890-SL (3) and Crafc0 903-SL (4) have outperformed their eastbound counterparts over the entire time span considered. The Dow 890-SL (3) section has maintained an effectiveness of at least 95% in the westbound lanes, yet its counterpart in the eastbound direction has deteriorated steadily to below 60%. The westbound Crafc0 903-SL (4) section has never dropped below 80%, yet the corresponding eastbound section began deteriorating quickly and never came close to matching the westbound performance. The eastbound section of Dow 890-SL (4) had better effectiveness values than the westbound section in early life, yet at approximately 25 months, it began to lose effectiveness very quickly and has since dropped below the latter. Additional surveys are needed to decipher the performance of these sealants over an extended period of time.

4.3 Pavement Structural Performance

Turning now to the observations regarding the structural performance of the pavement slab, the following remarks may be useful in summarizing the results obtained by the research team. Collection of data pertaining to PCC pavement performance was initiated during the Spring 2000 evaluation, after several mid-slab cracks had been noticed. Only the westbound driving lane had been included in this initial survey. During the Fall 2000 evaluation, the initial pavement performance survey of the eastbound driving lane and the second such survey of the westbound driving lane were conducted. Additional surveys were conducted during the Spring 2001 and Fall 2001 evaluations, during which the number of slabs containing transverse cracks and slabs with corner breaks were recorded. Slabs containing more than one transverse crack or corner break were counted as just one. The degree of joint spalling, measured by length, was calculated as well. These three pavement distresses are examined below.

Unlike sealant performance, in which the westbound lanes are superior to the eastbound, pavement structural performance in the eastbound lanes is higher than in the westbound lanes, judging by the corresponding frequencies of transverse cracking. As of the latest survey (October 2001), the westbound lanes have 44% of their slabs cracked, but the eastbound only have 39%. This is surprising since the eastbound lanes are approximately one year older than the westbound. The fact that the westbound lanes have superior sealants but more extensive transverse cracking suggests that no correlation exists between sealant effectiveness and transverse cracking; a closer inspection verifies this assumption. It is observed that many of the sealant sections that have high

effectiveness values also exhibit high percentages of transverse cracking. In addition, many of the sections with low effectiveness values have very little transverse cracking. Finally, the distribution of cracking is generally random, corroborating the assertion that no correlation exists between sealant effectiveness and transverse cracking.

There are only three sections with corner breaks in the eastbound lanes accounting for five slabs. Two of these sections are sealed with a compression seal and a silicone, respectively, whereas the third is unsealed. Therefore, it can be inferred that corner breaking has no correlation with sealant effectiveness, either.

The structural performance of the eastbound lanes suggests that the mere presence of a sealant may prevent spalling at the joints. Excluding a poorly constructed joint in the hot-applied section of Crafc0 221 (1), the two sections that contain the most amount of spalling are the two unsealed sections. Among the sealed sections, however, those containing the highly effective compression seals also exhibit the highest extents of spalling, suggesting that the effectiveness of a seal is not a guarantee against this type of distress. Yet, to complicate matters, the westbound lanes do not always exhibit the same pavement distress trends as the eastbound lanes.

The extent of transverse cracking in the westbound lanes is surprisingly high: almost half of the slabs are cracked. Obviously, the highly effective sealants did not prevent such cracking. The compression sealed sections, which generally are the best performing seals, have the top three rankings in terms of transverse cracking. The unsealed sections, on the other hand, have some of the lowest percentages of slabs cracked. It is apparent that excellent sealant performance does not promote good

pavement performance in the westbound lanes any more than it did in the eastbound direction.

There are more than twice the number of corner breaks in the westbound lanes as there are in the eastbound lanes. Corner breaks in the westbound lanes are distributed evenly among the sealant sections, which suggests that no correlation exists between sealant effectiveness and corner breaking. The degree of spalling in the westbound lanes may suggest a faint correlation with sealant effectiveness: the top four sealant sections in terms of sealant effectiveness have a total of only 51 mm (2 in.) of spalling.

4.4 Profilometer Surveys

At approximately the same time period that the sealant and pavement evaluations are conducted, surface profilometer surveys are performed by ODOT personnel. Data are collected in the driving and passing lanes in both directions by a profilometer van, which makes three passes in each lane. The data are later sent by ODOT to the University of Cincinnati research team for analysis. Included are three measures of pavement surface roughness calculated using a mathematical algorithm from relative surface elevation data collected using ODOT's K.J. Law Non-Contact Inertial Profilometer, Model 690DNC (Shahin, 1994). These measures are the left wheel-track International Roughness Index (IRI_{lf}), the right wheel-track International Roughness Index (IRI_{rt}), and the average of both values of International Roughness Index (IRI_{bh}). In addition to these indices, two supplementary sets of values are calculated, referred to as the Mays Number (MAYS) and the Present Serviceability Index (PSI). This terminology reflects the expectation that

these mathematically determined measures somehow simulate the corresponding conventional indices. The latter should be established instead using a suspension response vehicle, or with reference to road user panel ratings that have been correlated through statistical regression to measured pavement distresses, respectively.

Figures 4.4 to 4.7 show the long-term performance of the pavement surface, displaying the results of the past seven surveys. The profilometer surveys for all four lanes follow similar, if not identical, trends. Between the December 1999 and March 2000 surveys, nearly every index demonstrates a significant increase in smoothness, followed by an equivalent decrease during the following survey (October 2000). After the latter survey, the profile surface for the eastbound lanes is fairly constant, while the westbound lanes increase and decrease during the last two surveys. The remarkable similarity in the trendlines between all four lanes suggests that climatic factors, e.g., curling and warping, may be responsible for the fluctuations in the pavement profile, rather than pavement deterioration. If true, this would make it difficult to rely on future profilometer data to show deterioration in the pavement. Also, while analyzing the profilometer data on a section by section basis, it is apparent that no correlation exists between sealant effectiveness and pavement surface deterioration. Many of the superior sealant sections exhibit decreases in pavement surface smoothness, while many of the inferior sealant sections show increases in smoothness. Additional insights may be obtained by reviewing the profile data directly as recorded by the computer on board the profilometer van, but these are no longer available.

4.5 Sealant Conclusions

The deteriorating condition of the sealants in the eastbound lanes was first reported by Hawkins, et al. (2001), when that stretch of the pavement had been open to traffic for a little longer than a year. The majority of the hot-pour and of the silicone sections had “already experienced significant full-depth adhesion failure, with the sealant either sinking completely into the joint or being pulled away by traffic.” Consequently it was recommended that “serious consideration needs to be given to the joint cleaning and sealant placing operations currently employed,” while “the two most significant deficiencies” were identified as “the omission of sandblasting during placement and inadequate sealant recess.” It was also noted that “the joints in this experiment were cleaned only by water- and air-blasting, even though the manufacturers’ recommendations usually called for sandblasting. [The specifications from ODOT stipulate that sealants “shall be installed in accordance with the manufacturer’s recommendations.”] It is possible that the extensive adhesion loss already noted is related to the joint cleaning procedures. Sandblasting provides a rougher surface for the sealant to bond to, but even this may not be enough. The surfaces of the joints need to be subjected to inspection before sealing, to ensure that they are clean and free of moisture, as this is an important detail in obtaining effective, long-lasting sealed joints. If the equipment and procedures employed in placing silicone and hot-pour sealants during this experiment represent the conditions on a typical highway construction site, it is apparent that not sealing would have been a preferable alternative, in terms of convenience as well as cost.”

Another observation noted early during this project was that “the worst of the sealed sections were obviously those with a narrow joint width of 3 mm (1/8 in.). In most joints with such a configuration, the sealant material had overflowed and run onto the pavement surface, thereby being exposed to tire traffic. Oversight and inspection provided were ineffective in averting the use of equipment and procedures that were obviously inadequate. Special nozzles or applicators need to be used, so that the sealant will be placed from the bottom up at a slow rate, ensuring that the joints are not overfilled” (Hawkins, et al. 2001). The performance update reported in this thesis provides additional justification for all these observations.

Data collected and analyzed since these early observations were made, however, suggest that there is little—if any—correlation between sealant performance and the development of structural distresses or of surface roughness in the pavement.

Figure 4.1 Deterioration of sealants in the eastbound lanes to date

Note: Crafcro 221 and 444 were installed 4 months after the other westbound sealants

Figure 4.2 Deterioration of sealants in the westbound lanes to date

Note: Crafco 221 and 444 were installed 4 months after the other westbound sealants

Figure 4.3 Comparison of eastbound and westbound lane sealants after 3 years in service

Figure 4.4 Trendlines for the Eastbound Passing Lane through October 2001

Figure 4.5 Trendlines for the Eastbound Driving Lane through October 2001

Figure 4.6 Trendlines for the Westbound Passing Lane through October 2001

Figure 4.7 Trendlines for the Westbound Driving Lane through October 2001

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary and Conclusions

Pavement drainage systems remove infiltrating water as quickly as possible in order to prevent such detrimental effects as pumping of fines, slab faulting, frost heave and strength reduction in the base, subbase and subgrade. Typical drainage provisions include the placement of a free draining aggregate layer or blanket, as well as the installation of longitudinal and transverse drains. Drainage at the United States (U.S.) 50 test pavement is accomplished through the use of a 100-mm (4-in.) open-graded aggregate base course, a 100-mm (4-in.) longitudinal pipe underdrain, as well as transverse collector pipes, spaced at 152 m (500 ft) intervals, evacuating moisture out of the pavement system into adjacent drainage ditches. The ditches are primarily designed to transport storm water away from the pavement and into the nearby Hocking River.

During a routine evaluation of joint sealant condition performed in March of 2000, it was noted in the Hocking River, which is located approximately 23 m (75 ft) south of the highway, that water levels had risen significantly due to previous heavy and continuous rain events. In fact, the flood surface appeared to be only 1.5 to 3 m (5 to 10 ft) below the pavement surface in places. Water ponding was observed in culvert ditches located alongside the westbound lanes of the highway. Through capillary rise, this flooding event undoubtedly saturated the underlying layers of the pavement system, i.e., the subgrade, subbase and base soils. This moisture infiltration may have caused strength

reductions in these layers. Clearly, joint sealants are completely ineffective at preventing water infiltration from the bottom of the pavement system up. Such events may lead to the development of pavement distresses, regardless of the presence or condition of the joint seal.

Three detailed sealant inspections were made between October 2000 and October 2001. The sealants were visually inspected and rated on their ability to maintain a water-tight bond with the pavement joint. In general, the compression sealants are superior to the silicone and hot-applied sealant sections. An exception among the compression sealants is the TechStar W-050 (5) section, which performed very poorly almost from the very beginning of the experiment. The performance of the silicone and hot-applied sealants depends significantly on the lane direction in which they were installed. The westbound lane seals are observed to be performing much better than those in the eastbound lanes even when compared to the eastbound sealants at the same age. The westbound silicone sealants are outperforming the eastbound sealants at the same age by 47%. The same comparison for the hot-applied sealants yields an average difference of 29% in favor of the westbound sealants.

Unlike sealant performance, in which the westbound lanes are superior to the eastbound, pavement structural performance in the eastbound lanes is higher than in the westbound lanes, judging by the corresponding frequencies of transverse cracking. As of the latest survey (October 2001), the westbound lanes have 44% of their slabs cracked, but the eastbound only have 39%.

The profilometer surveys for all four lanes follow similar, if not identical, trends. The remarkable similarity in the trendlines among all four lanes suggests that climatic factors, e.g., curling and warping, may be responsible for the fluctuations in the pavement profile, rather than pavement deterioration. Also, while analyzing the profilometer data on a section-by-section basis, it is apparent that no correlation exists between sealant effectiveness and pavement surface deterioration. Many of the superior sealant sections exhibit decreases in pavement surface smoothness, while many of the inferior sealant sections show increases in smoothness.

Along with the sealant and pavement inspections, the University of Cincinnati research team conducted an evaluation of the underdrain outlets since they too play a pivotal role in the performance of the pavement. The outlets and outlet pipes were viewed without the assistance of any special borehole video equipment. By viewing the outside of the outlets, however, substantial evidence could be gathered to determine the condition of the outlet pipes in that area. Many of the outlets (27%) could not be found either due to high vegetation growth or simply because the outlet was not located per Ohio Department of Transportation (ODOT) specifications. Many of the outlets contained large amounts of silt and debris, so that water from the underdrains could not flow out. Some of the debris is due to the presence of rodent screens, which allowed moss to grow and trapped silt, creating a dam. None of the eastbound outlets contain rodent screens, but nearly all the westbound outlets do. Some of the outlets are completely dry; since there had been a large amount of rainfall during the previous day, it can be inferred that the pipe is either completely blocked further inside or severed, and

water is traveling a different route. A correlation between steeper outlet pipe gradients and lack of silt and debris is apparent. The steeper slope causes the water to discharge at a higher velocity allowing the silt and debris to be carried with it.

Calculations were performed using the software program “Drainage Requirements in Pavements” (*DRIP 2.0*) developed by the Federal Highway Administration (FHWA). The *DRIP 2.0* program is designed to provide the user with three essential outputs: permeable base thickness, edge drain outlet spacing, and separator layer/subgrade compatibility. It was determined through calculations that neither the eastbound nor westbound lanes were constructed with a sufficient open-graded base since 5 of the 6 calculations called for a base thicknesses about twice as large as those constructed at the project site.

Although edge drain outlet spacing varies with longitudinal slope it was determined that the test pavement was constructed with adequate outlet spacing. Typical spacing at the project site was approximately 152 m (500 ft).

The U.S. 50 test pavement was constructed without a separation layer but in actuality the 100-mm (4-in.) dense-graded subbase serves as a separation layer since it is the layer between the open-graded base and subgrade. This layer was found to be incompatible with most of the subsoils found at the project site; only gravel soils were found to be compatible with the separator layer.

Overall it is apparent the subsurface drainage system constructed at the U.S. 50 test pavement was designed with an insufficient open-graded base thickness according to

the *DRIP 2.0* computer software. Further, it is also apparent that portions of the test pavement were not constructed within the original specifications.

5.2 Recommendations

Based on the research and findings from this study, the following recommendations can be made at this time:

1. Implement a drainage outlet maintenance program that includes cleaning silt and debris from the outlets on an annual basis. This will allow the outlets to drain more freely, as was the case when the research team performed such cleaning during their inspection of the drainage features. The maintenance program would be aided by the presence of outlet markers, which would clearly show the location of the outlet so that maintenance personnel could find the outlet without much delay. During the Spring 2000 evaluation, large areas of ponded water were noted in the drainage swales alongside both directions of the highway, with water levels appearing to be almost at the elevation of the pavement subgrade surface. Additional investigations into the effectiveness of the open graded base material and the elevations and pitch of the drainage ditches should also be conducted.

2. Design future roadway projects involving permeable bases using the *DRIP 2.0* software program or any current means approved by the FHWA. Subsurface drainage parameters should not be chosen arbitrarily and efforts should be made to match the design with the local climatology and geology.

3. Remove and replace all sealants having an average effectiveness below 75%, and thus are ineffective at preventing water and incompressibles into the joint. This recommendation is based on the statements made by Shober (1997) warning of the dangers of partially sealed joints. Accordingly, sealants in the eastbound lanes, except for two of the compression seals, should be removed. The TechStar W-050 seals should be replaced with another compression seal such as Watson Bowman WB-687 or WB-812. Alternatively, the deteriorated sealants may be replaced only in the eastbound lanes, leaving the westbound lanes unchanged, for the purpose of a less expensive yet useful comparison, so as to possibly verify conclusions from studies elsewhere, notably in Wisconsin.

4. Monitoring of joint sealant and pavement performance should continue for at least another five years to collect long-term performance data. The performance evaluation plan developed by the University of Cincinnati research team and implemented in the five evaluations since Fall 1999 should be used for all future evaluations of joint seal condition (Ioannides, et al., 2001). This will provide consistent evaluations and will generate information that is reliable and comparable to that collected during future inspections. Performance monitoring of the sealed and unsealed test sections should continue under the established survey routine.

5. Monitoring of pavement surface roughness via profile surveys should continue for the purpose of affirming or clarifying the trends revealed to date regarding the comparative performance of sections in the eastbound and westbound lanes, and providing a record of the progressive deterioration of each section. Attempts should be

made to establish general roughness trends of the pavement, as well as to correlate seasonal curling and warping with roughness trends. The possibility of equipment malfunctions should be considered. There were several instances where data collected during the profile assessment was inconsistent and error plagued.

6. Pending the results of additional investigations into the effectiveness of sealants in the Wet-Freeze climatic zone, the use of compression seals, e.g., Watson Bowman and Delastic should continue. The use of TechStar W-050 should be discouraged as this material has been proven to be unsuitable for pavement applications.

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