THE OHIO HPCP JOINT SEALANT EXPERIMENT

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ABSTRACT

This Paper describes the design and construction of the Ohio U.S. 50 test pavement, a project intended to evaluate concrete pavement performance in connection with various sealant types and joint configurations in the Wet-Freeze climatic zone. Unsealed transverse joints are also included in the experimental factorial. Monitoring activities are discussed and the sealant performance to date is summarized. It is reported that significant mid-slab cracking has been observed in the test pavement, but that this distress appears unrelated to the performance of the sealant treatments. The latter retain an overall effectiveness between 40 and 75% in the eastbound lanes after two years of service, with compression seals outperforming silicone and hot-pour sealants.

INTRODUCTION

In 1992, a number of state, federal and industry pavement engineers from the U.S. participated in a tour of several European countries with the purpose of reviewing their practices for and experience in improving Portland cement concrete (PCC) pavement performance. In the aftermath of this tour, a program was formulated by the Federal Highway Administration (FHWA) for assessing the effectiveness of a number of innovative concrete pavement design and construction features. The ultimate aim of the program is the design and construction of high performance concrete pavements (HPCP). These pavements will be characterized by three attributes: incorporation of innovative construction and design features and materials; enhanced construction techniques that lead to increased productivity; and ride quality and prolonged service life, resulting in lower life cycle costs. The immediate goal of the HPCP program is to construct selected highway projects across the U.S. to investigate innovative PCC pavement design and construction concepts. The long-term objective is to improve PCC pavement performance through innovations and research into their design, materials, construction technology and equipment, as well as evaluation of promising pavement technology developments from other countries.

Fifteen projects have been approved for funding under the HPCP program since its inception in 1996, including three in the state of Ohio. All three Ohio projects, developed by the Ohio Department of Transportation (ODOT) in collaboration with the FHWA, are located along a stretch of reconstructed PCC pavement on U.S. 50, outside the city of Athens, Ohio. One of these projects is designed to evaluate PCC pavement performance in connection with various sealant types and joint configurations, including unsealed transverse joints. This Paper describes the design and construction of the U.S. 50 test pavement, together with the experimental design for the sealant investigation. Monitoring activities are discussed and the sealant performance to date is summarized, thereby providing an update to a prior report published by the Transportation Research Board (Hawkins, *et al.*, 2001).

THE U.S. 50 TEST SITE

The U.S. 50 test site consists of a 3.3-km (2.0-mile) stretch of a four-lane divided highway located along U.S. Route 50, approximately 1.3-km (0.8-mile) east of the city of Athens, in Athens County, southeast Ohio. The experimental pavement is part of a 10.5-km (6.5-mile) stretch of U.S. 50 under reconstruction. 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. In the higher elevations of Athens County, winters are cold and snowy, with a mean annual snowfall of 447 mm (17.6 in.). In the valleys, it is also frequently cold, but intermittent thaws prevent a long-lasting snow cover. 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). Construction of the U.S. 50 test site in the Wet-Freeze zone eliminates a gap in the on-going Strategic Highway Research Program (SHRP) Specific Pavement Studies (SPS)-4 experiment, which is investigating the effectiveness of various joint sealing techniques in different climatic regions across the United States.

The pavement system consists of a 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.).

PAVEMENT DESIGN AND CONSTRUCTION

The 1993 American Association of State Highway and Transportation Officials (AASHTO) design procedure for rigid pavements was used by *Parsons Brinkerhoff, Inc.* as contractor to ODOT in determining the required slab thickness. Expected 80-kN (18-kip) equivalent single axle loads (ESALs) over the anticipated twenty year design period of the pavement were estimated based on traffic survey data collected in 1991. At the start of the design period, the average daily traffic (ADT) count was 7820 vehicles. At that time, the percentage of trucks, T, in the ADT was 16%. The directional distribution factor, D, was assumed to be 50% for the analysis. The design year (2011) ADT was estimated to be 10,950. Interpolating between the 1991 and 2011 ADTs, the 20-year average (2007) ADT was determined to be 10,324. The U.S. 50 test pavement was given the functional classification rural principal arterial. Based on the information above, it was determined that the pavement would be subjected to approximately 11 million ESALs.

Design variables unique to concrete pavements include modulus of rupture, M_R, concrete modulus of elasticity, E_c, modulus of subgrade reaction, k, as well as the load transfer coefficient, J, and drainage coefficient, C. Values of E_c and M_R selected for the pavement design were 24.8 GPa (3,600,000 psi) and 4.8 MPa (700 psi), respectively. To characterize subgrade support, a k-value of 27 MN/m^3 (100 pci) was conservatively chosen to represent seasonal changes in the condition of the underlying soil and the impact it may have on design slab thickness. The load transfer coefficient is intended to reflect the ability of a concrete pavement to transfer load across joints and cracks. Due to the presence of tied concrete shoulders and dowel reinforced transverse joints in the pavement, a load transfer coefficient of 2.80 was selected. The quality of drainage and the duration of saturation levels in the underlying granular layers are reflected in the drainage coefficient. A coefficient of 1.0 was selected as appropriate for the drainage provisions at the test pavement, which include an open graded base layer. According to the AASHTO Guide, a value of 1.0 may characterize a material that has good to poor drainage and exhibits saturated moisture levels 1 to 25% of the time.

The level of reliability selected was 85.0%, with a standard deviation of 0.39. Initial and terminal serviceability indices used in the design equations were selected as a function of pavement type and construction quality. Based on the pavement surface texture and expected traffic volumes for the pavement, initial and terminal serviceability indices of 4.20 and 2.50, respectively, were chosen.

Construction of the test pavement occurred in two phases, the first involving the eastbound and the second the westbound lanes. Construction of the eastbound lanes began in the Summer of 1997 and these lanes were opened to traffic in Spring of 1998. Concreting and first sawing was completed in October 1997, while the secondary cut—where needed—was made in October and November, and sealing occurred in November. During this 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. Concrete placement occurred between the months of September and October 1998, and secondary joint sawing operations occurred in December 1998. By that time, only eight of the ten joint sealant types had been installed, but sealing was suspended due to low temperatures. The remaining two (hot-pour) sealants were not placed until April 1999, when the slab temperature was above the manufacturer’s suggested minimum for installation. The westbound lanes were opened to traffic in May 1999.

TEST SECTIONS

Test sections are the numbered portions of the highway pavement that encompass one of fifteen specific sealant material and joint configuration combinations, referred to as a treatments, for some distance or number of joints. For this experiment, the pavement is divided into thirty different test sections, which are typically 180 m (600 ft) in length, with approximately thirty transverse joints per section. In general, two replicate sections of each treatment were constructed, one in the eastbound and the other in the westbound lanes. One of the primary objectives of the experiment is to determine whether or not there is a distinct advantage in using one type of treatment over another as it relates to pavement performance. In the eastbound lanes of the project, the test sections are located between Stations 154+00 and 290+00, while those in the westbound lanes begin at Station 133+60 and end at 290+00. Transverse joints between Stations 231+00 and 260+00 in both directions are not included in the experimental design nor in the performance evaluations. This stretch corresponds to the location of the batch plant and of the headquarters of the project contractor (*Kokosing Construction Company, Inc.*), an area of intense and heavy truck traffic.

As constructed, 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: Crafco 903-SL, Dow 890-SL, Crafco 902, and Dow 888; Crafco 221 and Crafco 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. By combining the various sealant materials and joint configurations, a total of fifteen different treatments were formed. A detailed description of each sealant material and joint configuration installed at the U.S. 50 project can be found in Hawkins (1999), which also presents manufacturer supplied product literature in the accompanying appendix.

PERFORMANCE EVALUATION PLAN

The importance of continuous monitoring throughout all phases of the U.S. 50 sealant experiment has been recognized since the beginning of the project. Field notes have been kept and video records have been made during each stage of pavement construction, including subgrade preparation, PCC slab placement and joint sealant installation. Following the opening of the new pavement to traffic, the performance of the test sections included in the sealant experiment has been evaluated twice a year by the University of Cincinnati research team. Prior to the Fall of 1999, the University of Cincinnati research team had conducted two visual inspections of the eastbound lanes, as well as a single visual inspection of the westbound lanes (Hawkins, 1999, Hawkins, *et al.*, 2001). Since the project is concerned with the long-term performance and effectiveness of each joint sealant treatment, these early visual inspections provide an indicator of the initial condition, or early age performance. Information recorded is primarily in the form of visual observations made on three transverse joints in each test section, and performed by a research team member standing at the pavement shoulder. The joint sealant condition was described and visual estimates were made of the percentage of observed adhesive, cohesive or spall failures. Also noted was the depth to which the sealant was recessed below the pavement surface and the intrusion of any incompressible debris into the joint. From this information, conclusions have been made concerning premature aging and the relative rate of joint seal deterioration. It has been pointed out that "serious consideration needs to be given to the joint cleaning and sealant placing operations employed." More specifically, "the most significant shortcomings [at the U.S. 50 test site] appear to have been the omission of sandblasting at placement and inadequate sealant recess" (Hawkins, *et al.*, 2001).

In the Fall of 1999, the University of Cincinnati research team developed a methodology to be used in acquiring performance data in a consistent and organized fashion (Sander, 2001). Thus, a joint seal evaluation form was generated suitable for recording the types, extents and locations of failure and distress manifestations noted in each sealant, both numerically and schematically. The form includes the treatment type, the number and relative location of sampled joints, the beginning and ending stations, as well as measured distress and failure lengths, along with a legend of symbols used. This form was first used during the visual inspection of November 1999, and is to be used for all subsequent evaluations of joint sealant performance.

Because of the large number of transverse joints in each test section, which ranges from as few as 27 to as many as 126, it is necessary to devise a statistical sampling plan for performance monitoring. This allows investigators to evaluate a representative number of joint seals in each test section and to make inferences from these as to the condition of the entire section. To guarantee that no bias will be introduced into the results, the selection of a subset, or sample, is made on the basis of random sampling. The statistical sampling plan used for evaluations at the U.S. 50 project involves the examination of six randomly selected transverse joints in each of the thirty test sections. It is considered that a sample of size six combines the qualities of being large enough to be representative of the entire set, or population, while also being small enough to allow the evaluation of the test sections in two full working days by the available research project personnel. The same six joints in each test section will be evaluated throughout the duration of the experiment. The first, second, second to last and last joints in every test section were intentionally excluded from the selection process in order to eliminate possible overlap effects from adjacent sections.

The methodology developed for visual field inspections entails the following steps. Within each test section, six transverse joints are selected randomly for continual monitoring in accordance to a statistical sampling process described in a subsequent section of this Paper. Each joint selected is examined for signs of sealant failure and distress over a length of 1.83 m (6 ft), beginning at the outer shoulder joint and covering the right wheel-path of the driving lane. Each failure or distress type is identified according to a list of definitions and carried to the site by the inspector for instant reference. The length of any noticeable distress or failure is measured and recorded on the field evaluation form in the space allocated to that particular joint. The record includes a schematic indicating the position of each distress feature along the joint length surveyed. In the case of adhesive and spall distresses, the side of the joint, approach or leave, is also noted. These data collection activities follow closely the model established by similar investigations, primarily the SHRP studies performed by *ERES Consultants, Inc.* (Smith, *et al.*, 1999).

The lengths of each observed feature are summed to give the total failure length of that particular joint seal. Dividing the total failure length by the overall length inspected, i.e., 1.83 m (6 ft), the percent overall effectiveness can be determined for each joint. From these values, an average effectiveness figure is determined for each section, and a seal performance rating category is assigned to the section according to the scheme developed by Belangie and Anderson (1985). Sealants exhibiting effectiveness levels between 90 and 100% are classified as being in very good condition, whereas those sealants showing less than 50% overall effectiveness are in very poor condition and are considered to have failed. Performance ratings of poor, fair and good are assigned appropriately to sealants having effectiveness levels ranging between 50 and 90%. Such a system ensures that the performance and condition rankings assigned to each sealant are consistent between evaluations. It is noted that the same ranking scheme was also used during the Strategic Highway Research Program (SHRP) H-106 and Specified Pavement Studies (SPS)-4 experiments (Smith, *et al.*, 1999; Evans, *et al.*, 1999). Consequently, results obtained in Ohio will be directly comparable to those from other national studies.

ANALYSIS OF FIELD PERFORMANCE DATA

Treatment Effectiveness in the Eastbound Lanes

Quantitative data on joint seal effectiveness in the eastbound lanes in accordance to the aforementioned evaluation plan were first collected in November 1999. This data set is code-named EBNV99. In March 2000, the University of Cincinnati research team collected a second set of performance data in the eastbound lanes. The corresponding data set code-name is EBMR00. These observations are discussed in detail by Sander (2001). The EBNV99 data set indicates that the Watson Bowman WB-687 (Joint Configuration 5) treatment exhibited the highest overall effectiveness (97.8%). The worst performing treatment in this data set was the Crafco 444 (1), which exhibited a sealant effectiveness of only 14.4%. Compression seals, with the exception of the Techstar W-050 (5) treatment, were in very good condition, showing greater than 95% effectiveness. Both of the non-sag silicone sealant treatments, namely the Dow 888 (1) and Crafco 902 (1), showed poor performance, having less than 65% effectiveness. Results from the EBMR00 evaluation show that the Watson Bowman WB-687 (5) and the Delastic V-687 (5) treatments continued to exhibit very little deterioration, both having an overall effectiveness of 95.3%. With an effectiveness of only 9.7%, the Crafco 444 (1) remained the worst performing treatment. The other hot-pour section, Crafco 221 (1), experienced no deterioration over the four month period between evaluations, retaining an effectiveness of 71.9%. The section of Crafco 903-SL (4) between Stations 206+00 and 213+00 exhibited the largest deterioration, decreasing approximately 38 percentage points in effectiveness (from 62.5 to 24.2%), whereas the Crafco 903-SL (1) treatment declined by nearly 14 points (from 66.1 to 51.9%). The three Dow 890-SL silicone treatments (3, 4, 1) continued to show fair to poor performance, ranging between 55.0 and 67.8% in effectiveness.

Another way of evaluating the performance of experimental joint sealants is through analysis of deterioration over time. It is assumed that all treatments exhibited an effectiveness level of 100% immediately after installation. Deterioration is indicative of a sealant treatment's performance with time, and more importantly, of its longevity while maintaining a minimum acceptable level of effectiveness. At the time of the EBNV99 performance evaluation, the eastbound lanes had been exposed to traffic and weather for approximately twenty months. Of the four silicone sealants, the Dow 890-SL (1, 3, 4) treatments showed the best performance, exhibiting the lowest average joint seal deterioration over the four-month period between evaluations. Crafco 903-SL

(1, 4) treatments had the second lowest average deterioration at the time of the EBNV99 evaluation, yet deteriorated rapidly in the time period between the EBNV99 and EBMR00 surveys. Performance trends of the Dow 888 (1, 1) and Crafco 902 (1) silicone sealants indicate that their effectiveness has continued to decrease steadily over their approximate twenty four months of service.

The two hot-pour sealant treatments exhibited a significant difference in performance with age. Since installation, the Crafco 444 (1) treatment has shown a considerably faster deterioration as compared to the Crafco 221 (1) treatment. At the age of twenty months, Crafco 221 (1) was undoubtedly the better performing hot-pour sealant in terms of overall effectiveness, maintaining its resistance to environmental factors and traffic. Approximately twenty four months after installation, the Crafco 221 (1) sealant treatment continued to display a constant level of performance, whereas the Crafco 444 (1) deteriorated further, exhibiting a slight decrease in effectiveness over the four month period between evaluations EBNV99 and EBMR00.

Compression seals, with the notable exception of the Techstar W-050 (5) section, experienced minor deterioration over the twenty four month service period. Of the three compression seal sections in the eastbound lanes, the Techstar W-050 (5) treatment had the highest rate of deterioration, casting doubts concerning its long-term durability. In contrast, the other two sections exhibited excellent short-term behavior and are likely to continue to perform well in the future.

Deterioration rates of all three sealant classes installed in the eastbound lanes suggest that silicone and hot-pour sealant treatments deteriorated more rapidly than the compression seals. Compression seal treatments as a group outperformed silicone and hot-pour treatments by about 23 and 33 percentage points, respectively. Hot-pour sealants showed the highest rate of deterioration up to the age of twenty months. In contrast, their performance between twenty and twenty four months was relatively constant, showing very little joint seal deterioration over that time period. Unfortunately, the sealant had already deteriorated into very poor condition.

Each of the 13 sealed treatments may be ranked according to its level of overall effectiveness as of each of the two visual inspection surveys (EBNV99 and EBMR00). Additionally, depending on the percentage deterioration of each treatment in the four months between these inspections, a corresponding deterioration rank may be assigned. Note that a high rank is only desirable with regard to effectiveness, but not with regard to deterioration. The best performing sealant treatment is ranked as No. 1 in Effectiveness, whereas the worst performing one is ranked No. 13. In contrast, the most rapidly deteriorating treatment is ranked as No. 1 in Deterioration, whereas the treatment with the slowest or no deterioration is ranked No. 13. Information collected shows that at the time of the EBNV99 evaluation, the best and worst performing treatments in terms of overall effectiveness were the Watson Bowman WB-687 (5) and the Crafco 444 (1), respectively. In terms of deterioration rate, the Crafco 903-SL (4) treatment was ranked as No. 1 and the Techstar W-050 (5) as No. 2. Crafco 221 (1) treatment exhibited the least amount of deterioration between the EBNV99 and EBMR00 evaluations, earning the most desirable deterioration rank of 13.

These observations reaffirm the preliminary conclusions reached following the early inspections by the research team that “the worst of the sealed sections [are] those with a narrow joint width of 3 mm (1/8 in.). In most joints with such a configuration, the sealant material had overflowed... thereby being exposed to tire traffic... Special nozzles or applicators need to be used, so that the sealant will be placed from the bottom up at a slow rate, so that the joints are not overfilled” (Hawkins, *et al.*, 2001).

Treatment Effectiveness Westbound Lanes

At the time of the November 1999 inspection of the westbound lanes (dataset: WBNV99), four of the treatments, namely Dow 890-SL (1), Delastic V-687 (5), Watson Bowman WB-687 (5) and Dow 888 (1, Replicate a), showed no distress, having an overall effectiveness of 100%. In fact, ten of the thirteen sealant treatments were found to be in Very Good condition, with an overall effectiveness above 90%, and these included all three compression seal types. This may be explained by the relatively early age of these sections: at the time of the inspection, the westbound lanes had been exposed to traffic for less than six months. The Crafco 903-SL (1) and the Dow 890-SL (4) treatments had an overall effectiveness of 83.9 and 83.3%, respectively, i.e., they were in Good condition. In contrast, hot-pour sealant Crafco 221 (1) treatment exhibited an effectiveness of only 62.5%, and was the only treatment found to be in Poor condition at the time of the WBNV99 evaluation.

The largest decrease in effectiveness occurring in the four months between the WBNV99 observations and the March 2000 inspection of the westbound lanes (dataset: WBMR00) was recorded in the Techstar W-050 (5) treatment. Compression seals in this section showed a 29-point reduction in overall effectiveness, dropping from 98.3 to 69.7%. Several sealant treatments continued to remain in Very Good condition, all exhibiting less than a four percentage point decrease in effectiveness at the time of the WBMR00 inspection. These included both silicone sealants, Dow 890-SL (1), Dow 890-SL (3), Crafco 903-SL (1a) and Dow 888 (1a and b), and compression seals, Delastic V-687 (5) and Watson Bowman WB-812 (5). The latter treatment exhibited the smallest decrease in effectiveness, dropping from 100% to 99.7% between the two evaluations. The sealant treatment showing the worst performance was the Crafco 221 (1) hot-pour section. The overall sealant effectiveness of this treatment was 49.7% at the time of the WBMR00 inspection.

Treatments in the westbound lanes may also be ranked according to their overall effectiveness and rate of deterioration. Four treatments shared the No. 1 ranking for effectiveness at the time of the WBNV99 evaluation, namely, Dow 890-SL (1), Dow 888 (1a), Delastic V-687 (5) and Watson Bowman WB-812 (5). Following the WBMR00 inspection, however, only the Watson Bowman WB-812 (5) retained the honor of being No. 1, the other three treatments having fallen to the 4^{th}, 3^{rd} and 6^{th} spots, respectively. The Crafco 221 (1) treatment earned the lowest rank, No. 13, during both westbound lane evaluations. Over the four months between the WBNV99 and WBMR00 inspections, two of the three Dow 890-SL treatments, namely, Dow 890-SL (3) and Dow 890-SL (4) exhibited the smallest deterioration (dropping by less than 1 percentage point), gaining the desirable ranks of Nos. 12 and 13, respectively, for Deterioration. Eleven of the thirteen sealed treatments, including all eight silicone treatments and the Watson Bowman WB-812 (5), showed deterioration rates of fewer than 10 points over the four months between the two evaluations.

PCC PAVEMENT PERFORMANCE

To determine whether sealing transverse joints has an effect on concrete pavement performance, the sealant inspection plan calls for the recording of distresses occurring in the immediate vicinity of joints, and which may be indicative of joint seal inefficiency or failure. The first signs of such pavement distress were noticed on the first day of the EBMR00 evaluation, primarily in the form of mid-slab transverse cracks revealed in several of the test sections in the eastbound lanes as the wet pavement surface began to dry. The significant frequency and widespread distribution of these transverse cracks, however, did not suggest that their occurrence was necessarily related to the deterioration of any particular sealant treatment. Although their usual location at mid-slab was not as anticipated by the original sealant evaluation plan, it now appeared unjustifiable to simply ignore their presence altogether. Consequently, it was decided to conduct a pilot study into the frequency and distribution of transverse cracks, beginning with the evaluation of the westbound lanes the following day. Accordingly, all transverse cracks and corner breaks occurring in the driving lane over the entire length of the project were counted and recorded by section. It is anticipated that such observations will continue in both the eastbound and westbound directions during future evaluations.

Regarding the development of transverse cracks in jointed reinforced concrete slabs, Yoder and Witczak (1975) indicate that “the designer assumes a crack will form, generally at the center of the slab, and temperature steel is provided to keep this crack intact so that it will not open.” Similarly, Bradbury (1938) notes that “the strengthening or so-called ‘reinforcing’ of concrete members, through the medium of embedded steel, cannot be expected to actually prevent the concrete from cracking, since in any case—whether the structure be a building, a bridge, or a pavement—accomplishment of such a result would require the use of steel at such a low unit stress as to be decidedly uneconomical. Hence, the economical adaptation of reinforcing steel to any type of structure is fundamentally a problem of preventing what may be termed ‘objectionable’ cracking.” Monitoring of transverse cracks at the U.S. 50 test site, therefore, aims at assessing whether such cracks become objectionable from a functional viewpoint and, if so, whether this development is related to sealant performance in any way.

Transverse Cracking

During the WBMR00 evaluation, a distress survey of PCC slabs in the westbound driving lane of the Project, which stretches from Sta 133+60 and to Sta 290+00 skipping the slabs between Sta 231+00 and 260+00, was conducted. A total of 592 slabs were inspected and transverse cracks were observed in ten of the fifteen test sections. In some slabs, cracks had propagated across both the driving and passing lanes, whereas in others, cracking had been arrested by the longitudinal joint. Nearly every crack noted had developed at approximately the middle of the 6.4-m (21-ft) long slabs. The section displaying the greatest frequency of mid-slab cracks and the top percentage of slabs cracked was the one with the Dow 890-SL (1) treatment; a total of 9 cracks were noted, accounting for 33.3% of the 27 slabs. The section sealed with the Watson Bowman WB-687 (5) treatment showed the second highest percentage of cracked slabs, with 18.5% slabs cracked. The following sections exhibited no signs of mid-slab cracking at the time of the WBMR00 evaluation: Crafco 903-SL (1a); Dow 888 (1a); Crafco 903-SL (4); and Dow 890-SL (4). In addition, no transverse cracks were evident in the No Sealant (6) section.

When one considers that the majority of the joint seals in the relatively “young” westbound driving lane were in good to very good condition, it is rather unlikely that the transverse cracks observed in ten of the fifteen westbound test sections were related to poor joint sealant performance. Rather, it appears possible that structural factors may have been responsible for the premature cracking observed in a significant number of slabs. For this reason, a variety of pavement design features affecting pavement performance is discussed in a subsequent section.

Corner Cracking

Every transverse joint in the westbound driving lane of the Project was examined for evidence of corner cracking. There were no visible signs of corner breaks at any of the transverse joints in eight sections, including one that had unsealed joints. These are the two sections with the Crafco 903-SL (1) treatment, both sections with the Dow 888 (1) treatment, as well as the section of Watson Bowman WB-812 installed in joint configuration No. 5; the final unscathed section was the No Sealant (6) section. The other unsealed section in the westbound direction also fared quite well, exhibiting a single corner crack in one of its 125 slabs, accounting for 0.8% slabs cracked. The section with the Dow 890-SL (3) treatment had developed the highest percentage of slabs with corner cracks: four corner breaks were observed in its 28 slabs, accounting for 14.3% slabs cracked.

FEATURES AFFECTING PAVEMENT PERFORMANCE

Several key elements of sound pavement design are considered below in order to examine whether the pavement can continue to maintain high performance levels even if joint sealants were to deteriorate, allowing the infiltration of moisture and debris into the subbase, base and subgrade. Conversely, the probability that the pavement might deteriorate rapidly even if all sealants continued to function properly may also be assessed. A more detailed discussion of these and of several additional features affecting pavement performance is provided by Sander (2001).

Drainage

Drainage at the 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.

The design for the eastbound and westbound lanes of the test pavement called for the construction of a non-stabilized open-graded drainage base (NSDB), Item Special, placed in a single 100-mm (4-in.) lift directly beneath the 250-mm (10-in.) thick PCC slab (Item 451). The aggregate used for the base is an unbound crushed limestone. In the eastbound lanes, a “New Jersey” type NSDB satisfying the aforementioned specifications was placed, whereas in the westbound lanes, an “Iowa” type NSDB was used, because of its perceived superior long-term performance. Located between the subgrade and base is a blanket of granular subbase material, consisting of 150-mm (6-in.) of crushed aggregate (Item 304). To prevent the migration of silt- and clay-size particles into the overlying drainage base layer, the surface of the subbase was treated with a bituminous prime coat (Item 408), which was sprayed onto the surface of the compacted subbase and allowed to cure before placement of the base. Without this protective coating, the voids in the base might become clogged over time, thereby reducing or completely eliminating the drainage capacity of this layer.

Drainage design details for the test pavement called for the installation of longitudinal drains placed at the bottom of two trenches, one along the edge of the mainline PCC pavement slab and the other parallel to the outer edge of the shoulder. The outermost trench extended to a depth of approximately twice that of the drainage trench located below the PCC slab edge. The deeper trench primarily is intended to drain the subgrade, whereas the shallow trench is designed to evacuate water from the base and subbase layers. The trenches were excavated to a minimum width of twice the pipe diameter, or 205 mm (8 in.), and were lined with filter fabric underdrain wrap to prevent future clogging of the pipe. The filter fabric (Spec. 712.09, Type A) prevents fine-sized soil particles from entering the drain and choking the voids that would allow free passage of water. Granular material was used as backfill in the trenches and was placed to a minimum height of 300 mm (12 in.) above the top of the pipe. All longitudinal drains were constructed using a 102-mm (4-in.) diameter shallow pipe (Item 605) that was installed continuously as it was unwound from a large spool. The underdrain pipes were then connected with transverse outlets spaced at approximately 152 m (500 ft) intervals.

Despite the presence of these drainage features, extensive flooding occurred in March 2000, following several days of intense rainfall. To the south of the pavement, the Hocking River overflowed its banks, with the highway embankment itself serving as the river bank in many locations, where the water level rose to less than 1.5 m (5 ft) of the pavement surface. Extensive flooding was also observed to the north of the test pavement, covering several acres of farmland and woods. The pavement ditches disappeared under the flood pool and seemed unable to conduct the water under the pavement section and into the Hocking River for several days.

Joint Load Transfer

For the pavement-as-built at the U.S. 50 test site, load transfer across transverse joints is accomplished through regularly spaced epoxy-coated steel dowels. For the purposes of another experiment, these dowels are replaced at some of the joints by fiberglass bars or by stainless steel tubes filled with concrete. All dowels are 38-mm (1.5-in) in diameter and 460-mm (18-in.) in length, are spaced 305 mm (12 in.) on center and are supported on baskets located every 6.4 m (21 ft). To evaluate the effectiveness of this design, finite element computer program *ILSL2* (Khazanovich and Ioannides, 1994) is used to calculate stress and deflection load transfer efficiencies, as well as maximum values of deflection, bending stress, subgrade stress and concrete bearing stress. Adopting typical and reasonable values for the joint opening and the modulus of dowel reaction, calculated values of deflection load transfer efficiency range from 81 to 93%, while those for stress load transfer efficiency vary between 40 and 60%. Bearing stress values as high as 8 MPa (1150 psi) are obtained, the highest values being associated with improved load transfer efficiencies. This may result in concrete crushing under the dowel and may jeopardize the long-term effectiveness of the load transfer system.

Transverse Joint Spacing

Ioannides and Salsilli-Murua (1989) suggested that the spacing of transverse joints should be based on the non-dimensional ratio (L/*l*), of the slab length, L, to the radius of relative stiffness, *l*, of the slab-subgrade system, and recommended joint spacings corresponding to an (L/*l*) ratio ranging between 4 and 6 (with 5 being "a promising alternative"). Subsequently, on the basis of extensive field investigations, Smith, *et al.* (1997) recommended that in order to minimize transverse cracking in jointed plain concrete pavements, slab lengths should be designed such that the (L/*l*) ratio is less than about 4.5. The concrete pavement at the U.S. 50 test site is constructed with transverse contraction joints spaced every 6.4 m (21 ft). In order to assess the impact of this design on pavement performance, the (L/*l*) ratio may be calculated. A range of values, representative of materials at the test site, may be chosen for this purpose. Pavement design parameters noted above included a concrete modulus of elasticity, E_c, of 24.8 GPa (3,600,000 psi) and a modulus of subgrade reaction, k., of 27 MN/m^3 (100 pci) had been assumed. The corresponding (L/*l*) ratio using these values is approximately 6.1. Retaining the k-value noted, the (L/l) ratio is reduced to 5.3 when E_c increases to 41 GPa (6,000,000 psi). On the other hand, (L/*l*) values up to 7 or 8 are also within the realm of reasonable probability. Whether the amount of temperature steel reinforcement provided in the test pavement slab warrants exceeding the recommended (L/*l*) limit so much is rather debatable.

Tied PCC Shoulders

The new highway at the U.S. 50 test site incorporates tied PCC shoulders of variable width. The shoulders are designed with the same thickness as the mainline PCC slab, i.e., 250 mm (10 in.). On the outer side of the pavement (adjoining the driving lane), the shoulders are 3-m (10-ft) wide, whereas on the inner side (adjoining the passing lane), the shoulders are 1.2-m (4-ft) wide. The longitudinal shoulder joints are tied with 16-mm (0.625-in.) diameter steel reinforcing bars, 760 mm (30 in.) in length, and spaced every 760 mm (30 in.). In each slab, tie bars begin and end 305 and 457 mm (12 and 18 in.), respectively, from the transverse joints. A mechanistic analysis using *ILSL2* indicates that shoulder ties lower the free-edge bending stress by about 11 to 20%. Reductions in free-edge deflection range from 27 to 33%, whereas the free-edge subgrade stress is decreased by 26 to 33%. Thus, reductions in the stress and deflection levels experienced by the concrete slab on account of the presence of tied shoulders can be quite significant.

Reliability

The reliability level can be the most significant input parameter in the design because it defines the overall confidence level concerning the primary assertion of the engineer, i.e., that the pavement will serve applied traffic effectively during its projected life. A pavement engineer could produce a strong and economical design, yet a low reliability is certain to undermine confidence that the pavement will last its full design life. Although a lower level of reliability may be attractive because it dictates a thinner pavement slab, consideration of life-cycle costs associated with long-term maintenance often demonstrates the folly of seeking a lower initial cost in this manner. For highways with the functional classification of rural principal arterial, AASHTO recommends a design reliability between 75 and 95%, a range that encompasses the level of reliability selected in the design of the U.S. 50 test pavement.

Using the AASHTO design procedure, analyses are performed to study the effect of the selected reliability level on pavement slab thickness. It is found that upon increasing the reliability to 90%, the design slab thickness remains 250 mm (10 in.). Selecting a 95% level, however, yields a slab thickness greater than 250 mm (10 in.); for 99% reliability, the design slab thickness is over 280 mm (11 in.). Selecting such a low reliability level, therefore, makes the pavement more likely to experience early distress compared to a similar pavement designed using a reliability level of 95% or higher.

Construction Issues

Two pavement construction related issues may have contributed to the premature signs of distress, namely mid-slab transverse cracks, observed in both lanes of the highway, as well as a high degree of surface roughness (Present Serviceability Index values sometimes below 3.0), uncharacteristic of newly constructed pavements. These are the cold weather pouring of the PCC pavement slab and the use of ground granulated blast furnace slag in the mix design.

The PCC slab for the eastbound lane test sections was cast between October 16 and October 22, 1997, while concrete for the westbound test sections was placed from September 30 to October 7, 1998. National Climatic Data Center (NCDC) air temperature observations recorded from 10/16 to 10/22/97 for the area surrounding Athens, Ohio, show minimum and maximum daily temperatures of -4 and 19^{o} C (25 and 66^{o} F), respectively. In the westbound lanes, the minimum and maximum air temperatures recorded between 9/30 and 10/7/98 were 1 and 28^{o} C (34 and 83^{o} F), respectively. For such maximum daytime temperatures, the base was probably warm prior to being covered with concrete. As nighttime air temperatures approached and eventually fell below freezing on several occasions, the top of the newly placed concrete slab must have cooled excessively. This may have resulted in a large thermal gradient between the cold concrete surface and the warmer slab bottom, leading to upward curling during curing. Moreover, concrete placed and cured in cold weather may exhibit an increase in the time required to initial set, loss of durability and a slowed rate of strength gain.

For the purposes of a separate study at the U.S. 50 test site, the PCC pavement slab was constructed using a mix design in which 25% of the required Portland cement content was replaced with ground granulated blast furnace slag (GGBFS). Blast furnace slag is a by-product from the production of iron and primarily consists of silicates, alumino-silicates and calcium alumina silicates. When crushed or processed to cement fineness, slag has cementitious properties which make it a suitable replacement for Portland cement, and is usually substituted on a 1:1 basis. Use of GGBFS usually improves the workability of fresh concrete, yet at the same time decreases the water demand due to the additional paste volume. The use of slag cement in fresh concrete tends to retard cement hydration, thereby slowing the time to initial set and concomitant rate of strength gain. When compared to normal concrete, the presence of slag cement tends to slow early age strength development, but increases the ultimate strength after 28 days. The delay in setting time caused by the use of GGBFS, coupled with the cold weather conditions during curing, may have contributed to upward slab warping, compounding the curling gradient discussed above.

CONCLUSIONS

A performance report is presented for the U.S. 50 Ohio HPCP Joint Sealant Experiment, constructed near Athens, Ohio in the Wet-Freeze climatic zone. The experimental design, performance evaluation methodology and data analysis for this project conform to guidelines established by previous studies, particularly those under the SHRP SPS-4 sealing effectiveness initiative. The most recent sealant performance data presented were collected in March 2000, one and two years after the opening to traffic of the eastbound and westbound lanes of the test pavement, respectively (Sander, 2001). In many ways, these data follow the broad trends delineated by early performance inspections (Hawkins, *et al.*, 2001), and re-affirm that after two years of service in the eastbound lanes compression seals outperform all other sealed treatments, retaining an overall effectiveness of 75%. Silicone and hot-pour sealants exhibit overall effectiveness values of 50 and 40%, respectively. It appears likely that these general trends will be replicated in the westbound lanes as time goes by.

Mid-slab cracking of significant extent was first observed in both the eastbound and westbound lanes in the Spring of 2000, following an extreme flooding event, which inundated a extensive area on both sides of the highway embankment. Neither the drainage provisions in the pavement, nor the nearby Hocking River were able to handle the amount of precipitation received during this event. An examination of several features affecting pavement performance points to a number of possible mechanisms responsible for the development of premature mid-slab cracking. Prominent among these appear to be the long joint spacing and the low reliability factor adopted, as well as the cold weather pouring of the concrete slab, which includes 25% granulated slag. It is anticipated that pavement and sealant performance monitoring will continue for several years.

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