

**Research Report
KTC 95-18**

**PAVEMENT DISTRESSES
AT INTERSECTIONS**

by

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in cooperation with
Kentucky Transportation Cabinet

and

Federal Highway Administration
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August 1995



Commonwealth of Kentucky
Transportation Cabinet
Frankfort, Kentucky 40622

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June 14, 1996

Mr. Paul E. Toussaint
Division Administrator
Federal Highway Administration
330 West Broadway
Frankfort, Kentucky 40602-0536

Dear Mr. Toussaint:

SUBJECT: Implementation Statement KYHPR 92-144, "Pavement Distresses at Intersections" NCP Code 4C4B3292

Asphaltic concrete pavements at intersections and their approaches, where traffic is required to stop and start, exhibit several types of distress. Among the more prominent forms of these distresses are deep rutting, pushing and shoving, and severe washboarding. Meaningful amounts of funds allocated for maintenance operations are exhausted each year to rehabilitate intersection pavements that have become safety hazards as a result of simple traffic action. Significant savings may be realized if intersections and their approaches are designed and constructed to accommodate the shear stresses as well as fatigue to which they are subjected. The overall purpose of this study was to understand the factors that influence these distresses and determine procedures that may be implemented economically to significantly reduce the costly and repeated rehabilitation of intersection pavements. Several innovative techniques were investigated in attempts to accommodate the higher stresses realized at intersections and their approaches including Portland cement concrete, high-density plastic geogrids, and polymer-modified asphalts. A problem was noted concerning installation of these grids which will require additional attention before more extensive use of this strategy.

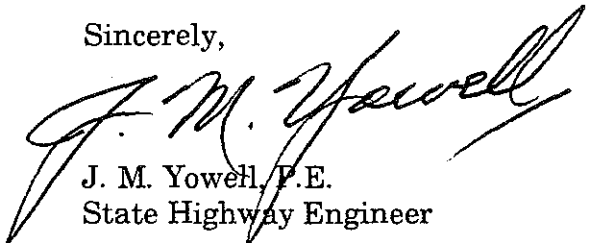
Based on the performance of the concrete sections observed during this study, rehabilitation with Portland cement concrete (PCC) pavement appears to be a preferred strategy for intersections which maintenance expenditures become excessive. The use of this strategy will also be encouraged for use at newly constructed intersections and approaches that are to be subjected to significant traffic volumes and loadings. Application of this strategy must be based on factors supporting the additional expenditures involved with this strategy. General guidelines will be developed and transmitted for your

Mr. Paul E. Toussaint
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information at a later date. The use of high-strength, polypropylene grids can be regarded as another way to reinforce an asphaltic concrete pavement to diminish the distressing effects of traffic such as rutting and shoving. High-density plastic geogrids were found to provide some resistance to long-term permanent deformation of asphaltic concrete pavement.

The Kentucky Department of Highways has utilized an extensive number of asphaltic concrete modifiers. However, there has been little, if any, follow-up with regard to performance monitoring. These experimental sections must be evaluated for long-term performance to document the effectiveness or ineffectiveness of the modifiers. The Kentucky Department of Highways will initiate an investigation to determine the types of modifiers that have been used within our state, the costs associated with their use, and the effectiveness or ineffectiveness of the applications. Information obtained from the investigation will be used by the Kentucky Department of Highways to develop warrants to achieve a proficient use of asphalt modifiers.

Sincerely,



J. M. Yowell, P.E.
State Highway Engineer

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16. Abstract Asphaltic concrete pavements at intersections and their approaches, where traffic is required to stop and start, exhibit several types of distress. Among the more prominent forms of these distresses are deep rutting, pushing and shoving, and severe washboarding. Prior research in this area has shown the leading causes of pavement failures at these locations are primarily materials related. Meaningful amounts of funds allocated for maintenance operations are exhausted each year to rehabilitate intersection pavements that have become safety hazards as a result of simple traffic action. Significant savings may be realized if intersections and their approaches are designed and constructed to accommodate the shear stresses as well as fatigue to which they are subjected. The overall purpose of this study has been to understand the factors that influence these distresses and determine procedures that may be implemented economically to significantly reduce the costly and repeated rehabilitation of intersection pavements. This report examines several innovative techniques used to accommodate higher stresses realized at these locations including whitetopping with Portland cement concrete, high-density plastic geogrids, and polymer-modified asphalts.					
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EXECUTIVE SUMMARY

Asphaltic concrete pavements at intersections and their approaches, where traffic is required to stop and start, exhibit several types of distress. Among the more prominent forms of these distresses are deep rutting, pushing and shoving, and severe washboarding. A severely rutted pavement not only increases a driver's discomfort but fosters poor public opinion about the condition of the pavement specifically and highway maintenance activities generally. Prior research in this area has shown the leading causes of pavement failures at these locations are primarily materials related. Meaningful amounts of funds allocated for maintenance operations are exhausted each year to rehabilitate intersection pavements that have become safety hazards as a result of simple traffic action. Significant savings may be realized if intersections and their approaches are designed and constructed to accommodate the shear stresses as well as fatigue to which they are subjected. The overall purpose of this study has been to understand the factors that influence these distresses and determine procedures that may be implemented economically to significantly reduce the costly and repeated rehabilitation of intersection pavements. This report examines several innovative techniques used to accommodate higher stresses realized at these locations including whitetopping with Portland cement concrete, high-density plastic geogrids, and polymer-modified asphalts.

Based on the excellent performance of the concrete sections observed during this study the use of Portland cement concrete is strongly recommended whenever maintenance expenditures at intersections are excessive. Further, PCC pavement should be considered for use at newly constructed intersections and their approaches. The use of high-strength, polypropylene grids can be regarded as one way to reinforce an asphaltic concrete pavement to diminish the distressing effects of traffic such as rutting and shoving. Observations indicate that high-density plastic geogrids placed in the asphalt layers provide some resistance to long-term permanent deformation. However, the key to using these high-strength materials efficiently for asphaltic concrete pavement reinforcement lies in the ability to suitably install the grid in a manner that it can provide the added tensile strength as designed. It is recommended that high-density plastic reinforcing grids be considered as a rehabilitation option if it can be determined that the geogrids can be suitably installed with minimal difficulties.

The Kentucky Department of Highways has utilized an extensive number of asphaltic concrete modifiers. However, there has been little, if any, follow-up with regard to performance monitoring of these experimental sections. These experimental sections must be evaluated for long-term performance to document the effectiveness or ineffectiveness of the modifiers. It is recommended that the Department initiate an investigation to determine the types of modifiers that have been used and locations of their use, costs associated with their use, the effectiveness or ineffectiveness of the application, and to develop warrants necessary to implement the use of only the most effective asphalt modifiers.

INTRODUCTION

Asphaltic concrete pavements at intersections and approaches, where traffic is required to stop and start, exhibit several types of distress. Among the more prominent forms of these distresses are deep rutting, pushing and shoving, and severe washboarding. These distresses cause significant safety problems for drivers, especially during wet weather driving. Safety problems may also occur during dry weather due to reduced skid resistance as a result of flushed pavements and roughness. Meaningful amounts of monies allocated for maintenance operations are exhausted each year to rehabilitate intersections that have become safety hazards because of the effects of static and dynamic traffic action at intersection approaches. Undoubtedly, intersections and approaches that are properly designed and constructed could be virtually free from these maintenance considerations for many years. Smoother pavements at intersections and approaches also greatly enhance the complete safety of the transportation system. This report examines the causes of deep ruts, pushing, shoving and severe washboarding at intersections and details present strategies being used to overcome these pavement distresses. Satisfactory resolution to these continuing problems will save lives and greatly reduce maintenance expenditures.

Pavements at intersections and approaches are subjected to a greater variety of loading conditions than for open road sections. Vehicles slow, stop, idle, and start at intersections. Structural designs for pavements and paving materials were developed for pavements having moving loads without consideration for the high repetitive shear at locations where vehicles stop and start. Additionally, increases in wheel loads and tire pressures in recent years have led to even further increases in shear stresses at these locations and at other nontangent segments of pavements such as curves and steep grades. Asphaltic concrete pavements are particularly vulnerable to the accumulation of fatigue distresses which lead to rutting, pushing and shoving, and severe washboarding. Severely rutted pavements present a safety hazard to the motoring public and can tax a driver's ability to safely traverse the roadway. A recent report issued by the Kentucky Transportation Center identified roadway related tort liability cases from 1981 through 1994, [1]. Of the total number of cases, there were 27 submitted to the Kentucky Board of Claims that indicated hydroplaning or water pooling (rutting) as a direct cause of the accident. Of those 27, only two accidents apparently occurred at intersections. A severely rutted pavement not only increases a driver's discomfort but fosters poor public opinion about the condition of the pavement in particular and about the highway department in general.

Prior research in this area has shown the leading causes of pavement failures at intersection locations are primarily materials related, [2]. The asphaltic concrete mixtures generally have been observed to contain asphalt contents in excess of the design value, relatively high percentages of natural (uncrushed) sand, and low voids in the mineral

aggregate. Intersections and other nontangent segments of a highway exhibit extreme forms of distress much sooner than tangent segments of the pavement and long before the design life of the pavement is reached. As a result, these pavement sections require maintenance and/or rehabilitation early in the pavement's service life. This early maintenance is costly in terms of materials and labor. Highway user costs are affected by these maintenance activities. Significant savings may be realized if intersections and approaches are designed and constructed to accommodate the shear stresses as well as fatigue to which they are subjected. Therefore, the overall purpose of this study has been to understand the factors that influence these distresses at intersections and determine procedures that may be implemented economically to significantly reduce the costly and repeated rehabilitation of pavements at intersections. The purpose of this report is to document current pavement distresses at intersections and approaches and to identify successful design solutions being used to accommodate the higher stresses realized at intersections and their approaches.

BACKGROUND

The objectives of this study are:

- A. Conduct a literature review to determine current attitudes relative to pavement distresses at intersections and approaches and to identify successful design solutions used to accommodate higher stresses at these locations. Develop recommendations, using information gained from the literature review concerning materials specifications and pavement design strategies, to construct trial pavements having sufficient strength and stability to withstand the shear forces encountered.
- B. Identify, through maintenance histories or other records from District personnel, intersection and approach pavements that have a history of early excess fatigue and short-interval maintenance repairs. The more severely distressed intersections will be nominated for trial installations through rehabilitation or replacement. Also, intersections and approaches that have not exhibited early signs of distress but are of equivalent design and have similar traffic histories will be identified. Possible reasons for this occurrence will be investigated.
- C. Evaluate construction techniques and overall pavement performance (as a function of time) of test sections constructed using a variety of rehabilitation

or replacement schema and compare the short-term performance to historical performance information (maintenance histories).

- D. Perform economic analyses to determine the most favorable strategies for rehabilitation or replacement of intersections and approaches.
- E. Develop a plan to implement the findings and results of this study and to incorporate possible designs, procedures, specifications, materials acceptance criteria, and construction techniques that may be developed in this study.

The successful completion of this research study was dependent upon accomplishment of objectives outlined herein. A comprehensive literature search was conducted using the facilities of the University of Kentucky Transportation Center Library. A listing of related articles/reports and accompanying abstracts was obtained and pertinent articles were acquired and reviewed. The detailed literature review documented state-of-the-art techniques relative to special considerations, materials specifications, pavement design considerations, and construction of pavements having sufficient strength and stability to withstand the vertical and horizontal shear forces encountered at intersections and approaches. A summary of the literature review is presented in this report.

Task B of the work plan involved identifying a limited number of prematurely distressed pavements within the twelve highway districts across Kentucky. A questionnaire was distributed to the highway Districts requesting that operations engineers nominate for possible further study two or three intersections or approaches that demonstrated significant premature pavement-distress related problems, such as deep rutting and severe washboarding. The questionnaire also requested engineers to identify one or two intersections or approaches that, historically, had not exhibited excessive pavement distresses early in the design life of the pavement despite being of similar design and construction and carrying similar traffic volumes and loadings. The engineers were instructed to limit their nominations to pavements that had been constructed using current mix design procedures [Special Provision No.43D (88)]. Division of Maintenance personnel within each District could identify problem intersection or approach pavements and provide valuable historical information relative to maintenance activities and maintenance intervals. Historical information relative to traffic characteristics, mixture design data, materials descriptions, typical sections from as-built plans, and other appropriate information for the identified intersection and approach pavements would be obtained from the District's personnel for Planning, Materials, Construction, and Design. Information relative to maintenance intervals, traffic accumulations, original pavement thickness design, mixture design, and present conditions of the nominated intersection and approach pavements was desired. Analysis of these data would provide insight as to why pavements

at some intersections and approaches apparently perform satisfactorily while others, although of similar geometric and structural design and experiencing similar vehicular volumes and loadings, had not performed as expected. Analysis of these data would also allow researchers to establish criteria to rank the performance of the nominated intersection and approach pavements. The performance ranking would allow selection and investigation of intersection and approach pavements for further study. The investigation may include, but not be limited to, performing visual surveys of the intersections and approaches; determining rut depths; observing the length and severity of washboarding from the intersection; obtaining field samples for laboratory analysis to determine relative air void content, asphalt content, aggregate gradation, and modulus values of the asphaltic concrete using creep tests; deflection testing and evaluation; and, in-situ evaluation of subgrade strength.

Task C of the work plan entailed implementation of experimental rehabilitation or replacement strategies. It was anticipated that four to six intersections or approach pavements would be examined initially. A variety of rehabilitation and/or replacement schemes were envisioned to be optimized, based on Kentucky's conditions and design procedures, and presented to the Kentucky Transportation Cabinet for consideration. Materials design, pavement design, traffic control considerations such as detours and delays, and any special construction techniques used while performing the rehabilitation or replacement would be documented. Intersections and approaches replaced or rehabilitated with Portland cement concrete would be evaluated for construction comparisons and for cost analyses. In as much as possible, a control section utilizing a conventional asphalt mixture would be evaluated for construction and performance comparisons, and cost analysis.

Core specimens would be obtained from pavements in which bituminous materials are used for pavement rehabilitation or replacement. The cores would be obtained as soon after final compaction as practical and before opening the intersection to traffic. Samples of the asphaltic concrete mixture would also be obtained and compacted in the asphalt plant laboratory at the specified field compaction temperature and density. Then, all specimens would be tested in the laboratory for their rutting potential. This exercise would provide a relationship for the rutting potential between field cores and laboratory compacted specimens. Specimens would also be evaluated for shear strength and modulus properties. In addition, air void content, asphalt content, and aggregate gradation of laboratory specimens and field cores would be evaluated. These evaluations would consist largely of comparing the values to determine if the method of compaction greatly affects these parameters. Also, cores would be obtained after the mixture has been in service for approximately two years. These cores would be tested for air void content, asphalt content, asphaltic concrete modulus determined from creep tests, and aggregate gradation. The

values obtained from these evaluations would be compared to initial test values obtained from field cores and laboratory specimens. These comparisons will yield an indication of the rate of change of the material properties with traffic loadings.

Performance evaluations included, but were not limited to, determining rut depths at variable distances from the intersection; determining the length from the intersection that washboarding begins and severity of the washboarding; evaluating longitudinal movement of the pavement; monitoring traffic characteristics such as vehicle volumes and loadings; and, performing profilograph tests of the approaches and intersections. Because of the limited time of this study, it was proposed that upon completion of the study, the performance evaluations would be continued under the long-term monitoring program to further document the rehabilitation or replacement strategies used.

Task D of the work plan considered economic analyses of the different strategies on both a first-cost and life-cycle basis. Costs associated with maintenance rehabilitations would be developed based upon results from the questionnaire regarding maintenance intervals for given intersections and approaches. These costs would include materials, equipment and labor costs.

Task E of the work plan involved developing recommendations relative to design procedures, specifications, materials acceptance criteria, and construction techniques which could be used to lengthen the service life of an intersection pavement and decrease maintenance needs based upon information gained during this study and presented for consideration for implementation.

LITERATURE REVIEW

The literature search focused on pavement distresses at intersections and methods used to upgrade intersection pavements. The methods most commonly used included: concrete overlays of asphaltic concrete pavements, or whitetopping; geogrid reinforcement of asphaltic concrete pavements; polymer-modified asphalts; stone mastic asphaltic concrete; and big-stone mixtures. The literature search also followed developments of the Strategic Highway Research Program relative to the evolution of performance-based asphaltic concrete mixtures.

Concrete Overlays of Asphaltic Concrete Pavements

The practice of placing a Portland cement concrete pavement over an existing asphaltic concrete pavement is commonly referred to as whitetopping. The initial use of whitetopping apparently dates back to 1918 when an existing flexible pavement in Terre Haute, Indiana, was overlain with 76 to 102 mm (3 to 4 in.) of reinforced concrete. After eight years of service, the concrete overlay was reported to be in excellent condition, [3]. Since that initial demonstration, several concrete types have been used to overlay existing flexible pavements including plain, reinforced, continuously reinforced, and prestressed concretes. According to Hutchinson, considerable use was made of plain concrete to resurface existing flexible pavements at both civilian and military airfields as a result of increased aircraft loadings during the 1940's and 1950's. Typical thicknesses of these overlays ranged from about 203 to 457 mm (8 to 18 inches). The US Army Corps of Engineers monitored the performance of the resurfacings at most military airfields and reported extremely good performance after 30 to 40 years, [3].

The level of service of airport pavements, interstate highways, primary and secondary roads, and parking lots has significantly increased as a result of the placement of concrete over existing asphalt. Portland cement concrete has been proven to be a stronger, longer lasting surface than asphaltic concrete pavements. Concrete pavements are not subject to rutting and shoving and, as such, are much more safer in wet-weather driving situations, [3]. Other advantages of whitetopping include avoiding possible reconstruction difficulties because the concrete is most often placed directly on the asphaltic concrete surface. Savings in construction costs are possible when concrete can be placed without removing or repairing the subbase or subgrade. However, performance of a concrete pavement is dependent upon the subgrade conditions, the condition and thickness of the existing pavement, and the thickness of the concrete overlay.

Hutchinson states that it is not unreasonable to expect a service life of at least 20 years, and even more, for concrete overlays. A properly designed and constructed whitetopped pavement will maintain a high level of serviceability over its life cycle. Serviceability is generally considered to be an assessment of the ability of a pavement to serve vehicular traffic and is a function of structural performance and rideability. Whitetopping an asphaltic concrete pavement improves the overall structural integrity of the pavement. Cracks and other distresses will eventually reflect through an asphaltic concrete overlay. However, because a properly designed and properly constructed concrete pavement can bridge over distressed areas, it will prevent structural related distresses from occurring. Information presented by Gisi in analyzing a project for the Kansas Department of Transportation indicated a much shorter life cycle for flexible overlays, [4]. Whitetopped pavements maintained their serviceability longer and required no significant maintenance.

For the design of a concrete overlay, the same procedures used to determine the thickness of a new concrete pavement can be used, [3]. Minimum slab thicknesses are based on expected traffic levels and underlying support values of the existing pavement structure. The foundation support may be determined by using a non-destructive test such as the falling weight deflectometer, a plate bearing test, or by estimating the modulus of the foundation support. The foundation support modulus may be estimated through the use of charts and graphs. Hutchinson provides information relative to the process some agencies use to determine the minimum thickness of a concrete overlay, [3]. Typically, recommended thicknesses range from 203 to 305 mm (8 to 12 in.) for interstate and primary routes. On secondary routes, thicknesses range from 127 to 178 mm (5 to 7 in.), although Iowa has used thicknesses of 102 mm (4 in.) on some county highways. Either a standard or conventional concrete mixture or a high early strength concrete mixture may be used to whitetop an asphaltic concrete pavement. The advantage of using a high early strength concrete mixture to whitetop a worn and distorted asphaltic concrete pavement is minimizing the disruption to traffic. Standard concrete placement, finishing and curing procedures are followed during a whitetopping operation. Transverse and longitudinal joints should be sawed as soon as possible to relieve initial stresses in the concrete overlay. Considerations for deeper than normal saw cuts should be given in instances where, due to distortions in the asphaltic concrete pavement, the nominal thickness of the concrete slab varies by more than 25 mm (1 in.). The pavement joints should be sealed with a high quality sealant in accordance with standard procedures to prevent moisture and incompressible materials from infiltrating into the pavement joint. Transverse joint spacings are typically the same for a concrete overlay as they are for newly constructed concrete pavements. Standard design considerations also apply to the use of load transfer devices, that is, conditions of support, traffic volumes and loads, and slab design.

There are three methods typically used for whitetopping an asphaltic concrete pavement; direct placement, milling the surface to produce a uniform profile prior to placement, or utilizing leveling courses to produce a uniform profile prior to placement. If there are no serious distresses in the asphaltic concrete surface, then the concrete overlay may be placed directly on the existing asphalt surface. Direct placement is cost effective in that there are minimum surface preparations involved. Direct placement is typically recommended for all instances where rutting in the asphaltic concrete pavement is 51 mm (2 in.) or less because rutting at these depths does not cause excessive thickness variations in the PCC overlay. However, cross sections must be taken by engineers to determine the volume of concrete required for the overlay, [5].

Typically, about five elevation readings are taken at each cross-section station. A cross-sectional area of the overlay is calculated at predetermined stations using the measured elevations and the planned surface elevation of the concrete overlay. These calculations are similar to those used to determine earthwork quantities.

Where rutting is greater than 51 mm (2 inches), a profile milling or planing machine may be used to mill surface distortions. Milling or planing machines are also used in those instances where geometric, vertical clearance, or right-of-way constraints prevent raising the final surface elevation. The existing asphaltic concrete surface is milled to a depth sufficient to place the concrete inlay within the geometric constraints of the site. The asphaltic concrete shoulders can be left in place and used as the trackline for the concrete paving machine. Also in instances where rutting is greater than 51 mm (2 in.), a leveling course may be used to remove distortions to create a uniform paving surface. However, placement of a leveling course may be more costly than milling or direct placement. One state conducted a study of these methods and found that using a leveling course cost nearly three times more than costs associated with milling and direct placement, [6].

Westall presented design and construction guidelines for concrete overlays of asphaltic concrete pavements, [7]. Based on ten years of accumulated experience, Westall indicated that because of the heat-retention properties of asphalt, the temperature of the asphaltic concrete surface had to be lowered prior to the placement of the concrete mixture to help prevent the concrete from setting too quickly, to preclude dangerous temperature gradients from occurring from the bottom of the concrete overlay to the overlay's surface and, also to hinder possible shrinkage of the flexible pavement due to the cooling effects of the concrete mixture. Westall stated that wetting the asphalt pavement for several hours prior to placement of the concrete overlay and nighttime paving were two ideal remedies to this problem.

Plain concrete overlays have been employed since the 1960's in California, Iowa, and Utah. Lokken reported that concrete overlays of flexible pavements had provided as much as 20 years of excellent service in California, [8]. The state of Iowa has successfully used plain concrete overlays since 1977 in rehabilitating their county road system. Schnoor and Reiner reported that concrete overlays of flexible pavements were in excellent condition in Iowa, [5]. The state of Utah has also made extensive use of concrete overlays on both I-84 and I-80 and have achieved excellent performance, [3]. The overall performance of whitetopping projects throughout the United States appears to be very good.

During September of 1991, an experimental project was initiated in Louisville to document the use of ultra thin concrete overlays of a flexible pavement, [9 and 10]. In what was truly a concrete industry effort, more than 40 groups pooled their resources and expertise to examine the construction and performance of very thin concrete overlays. A thin concrete overlay is considered to be less than 102-mm (4-in.) thick. The Louisville project utilized two overlay thicknesses, 89 mm and 51 mm (3.5 in. and 2.0 in.), with each section being 83.8 m (275 ft.) in length, to overlay an entrance road to Waste Management Corporation's Outer Loop Recycling and Disposal facility. The site offered several advantages to the groups

studying the thin concrete overlays. As many as 600 trucks per day use the route five and one-half days per week. All of the trucks were weighed upon entering and leaving the facility and the exact amount of loading the pavement experienced could be determined. This accelerated loading provided researchers with many years of vehicle loadings in only a few weeks time before the concrete overlays failed.

The existing bituminous pavement was milled on Friday, September 20, 1991, to provide a uniform overlay thickness. Fast-track paving techniques were employed because of the hours of operation at the site. The facility closes at midday Saturdays and reopens at 6:00 am on Monday mornings. The entire project had to be completed within that time frame. The concrete mixture was specially proportioned for high-early strength and contained 277-kg per cubic meter (800-pounds per cubic yard) Type I cement, a type F high-range water reducer, and polypropylene fibers at one kilogram per cubic meter (3 pounds per cubic yard). The fibrillated, 19-mm (3/4-in.) fibers were added to help control cracking. Paving started at about 2:00 pm on Saturday, September 21. A slipform paver was used to place the 7.3-m (24-foot) roadway in one pass. The surface of the milled asphalt pavement was dampened prior to dumping the concrete in front of the paver. A small amount of hand finishing was required and a broom finish provided the final surface texture. A white pigmented curing compound was applied to the thin concrete overlay. Paving was completed around 6:00 pm and sawing of the control joints began at 8:00 pm. The control joints were sawn at 1.83-m (6-foot) spacings and at a depth of 1/4 of the slab thickness. The roadway was opened to traffic at 6:30 am on Monday, September 23, 1991. After 38 weeks of continual usage, the pavement had experienced nearly 425,000 equivalent 8,165-kg (18,000-pound) axle loads. Typical low-volume roads and some city streets are designed to carry 50,000 to 100,000 equivalent 8,165-kg (18,000-pound) axle loads. Engineers evaluating the project surmised the superior performance was due to the bond developed between the concrete overlay and the asphaltic concrete surface even though no bonding agent was utilized. The bond between the asphaltic concrete and the Portland cement concrete enabled the asphaltic concrete layer to take a significantly higher proportion of the stresses thereby reducing the overall stresses in the concrete layer. Although the failed ultra-thin concrete pavement was replaced with asphalt, the Louisville project demonstrated the promise of ultra-thin concrete overlays.

Overall, the performance of concrete overlays of asphaltic concrete pavements has been excellent. The design of a concrete overlay employs the same procedures used to determine the thickness of a new concrete pavement. Minimum slab thicknesses are based on expected traffic levels and underlying support values of the existing pavement structure. Oftentimes, the concrete overlay may be placed directly on the existing flexible pavement. If pavement distortions exceed 51 mm. (2 in.), planing or milling may be used to create a uniform paving surface. Only those areas that have serious deterioration need be repaired prior to placing

the concrete overlay. Using an existing flexible pavement for support strengthens the overall pavement structure. The bond between the asphaltic concrete and the Portland cement concrete enables the asphaltic concrete layer to take a significantly higher proportion of the stresses, thereby reducing the overall stresses in the concrete layer. A whitetop concrete overlay significantly increases the safety of a pavement surface by eliminating rutting.

Geogrid Reinforcement of Asphaltic Concrete Layers

The concept of tensile reinforcement of asphaltic concrete layers has existed for many years and has been attempted with various materials. The idea is to reinforce the flexible pavement to increase tensile strength and extend the life of the pavement. Also, if greater tensile strength is available, then reduced thicknesses of asphaltic concrete materials are possible thereby resulting in an overall cost savings. The predominate materials for tensile reinforcement of asphaltic concrete layers during the 1950's and 1960's were welded wire fabric and expanded metal. Their use was attempted in the United States, Canada and in the United Kingdom [11, 12, 13, and 14]. These applications generally experienced installation problems in getting the welded wire fabric to lay flat and subsequently, the performance proved questionable. However, researchers did observe that when correct installation procedures were used, rutting depths were reduced when compared to a conventional pavement structure.

Tensile reinforcement became a much more potentially viable option with the introduction of TENSAR[®], a high-strength polymeric grid, in the late 1970's and early 1980's. Geogrids, when included into either an aggregate base layer or an asphaltic concrete layer, form a geocomposite which has greater stiffness and stability than aggregate alone, [15]. This can reduce both the magnitude of the vertical compressive strain on the top of the subgrade and the magnitude of the radial tensile strain at the base of the asphaltic concrete layer. A preliminary investigation of TENSAR[®] indicated the potential of the material for tensile reinforcement of asphaltic concrete layers. A large scale cooperative research program between the Ministry of Transportation and Communications of Ontario, Gulf Canada Ltd, Royal Military College and the University of Waterloo, was initiated shortly thereafter, [16, 17, 18, and 19]. Some of the early work involving the use of high-strength polymeric geogrids was reported by Brown, Haas, and Kennepohl, [20, 21, 22 and 23]. Some of the key findings of the laboratory investigations were that rutting depths were reduced up to 50 percent when a grid was placed within an asphalt layer, the number of load applications to a given rut depth increased by a factor of three when a grid was used, the effective thickness of the asphaltic materials could be reduced approximately 36 percent or the fatigue life

expectancy of a pavement could be increased by a factor of three with the use of the high-strength polymeric grid.

Based on the promising outcomes of the laboratory trials, several field trial projects were constructed throughout the United States, Canada, and Europe. The purpose for the field trials was to develop tensioning equipment, and installation and construction procedures. Initial installation procedures for reinforcement of asphalt layers with TENSAR® involved tensioning the grid along the pavement and fixing the grid to the surface by applying a chip seal or a slurry seal. After the binder hardened, the tension was removed, excess chips swept away and paving proceeded, [21].

Hughes documented the performance of some of the field trials in a paper presented at the Sino-British Highways and Urban Traffic Conference in November 1986, [24]. Hughes stated that conclusions reached during the laboratory trials had been substantiated during the field trials. The field trials established pavements having improved resistance to permanent deformation, fatigue cracking, and reflection cracking. Hughes estimated that the lifetime of a reinforced asphaltic concrete pavement may be increased ten times with respect to fatigue life and three times with respect to reflection cracking and rutting. Hughes indicated the site case histories had demonstrated the success of the installation procedures for TENSAR® grids. However, a 1990 article by Buist indicated that some early failures had occurred relative to the use of geogrids in asphaltic concrete layers that were generally attributed to faulty installation, [25]. The grid was observed to lift from the underlying asphalt layer where close contact was not made over the entire length of the grid. This resulted in insufficient cover of the grid and caused potholing.

The Georgia Department of Transportation's Office of Materials and Research conducted a study to evaluate the performance of TENSAR® AR-1 as overlay reinforcement for flexible pavement, [26]. The evaluation was conducted by comparing asphaltic concrete overlay pavement performance results of a control section with no reinforcement to the results of a section containing TENSAR® AR-1 reinforcement. The existing structure consisted of a 51-mm (2-in.) asphaltic concrete pavement over an unstabilized soil subgrade. The test section was constructed by placing TENSAR® AR-1 over the existing pavement and overlaying it with 51 mm (2 in.) of E-mix asphalt. Apparently there were no problems related to the installation of the TENSAR® geogrid. After a three year monitoring period, researchers concluded that the TENSAR® geogrid did not appear to prevent or reduce pavement cracking and rutting any better than where no pavement reinforcement was used.

A July 1992 article in *Better Roads* detailed the use of geogrids in the states of Texas, Maine, Minnesota, and New Mexico, [27]. The Texas Department of Transportation evaluated the use of geogrids to remedy rutting and shoving of asphaltic concrete pavements

on Interstate 10, 24.1 km (15 mi.) east of El Paso. The existing section included 152 mm (6 in.) of soil cement, 152 mm (6 in.) of flexible base, 76 mm (3 in.) of asphalt stabilized base and asphalt surface course. Researchers evaluated a variety of rehabilitation techniques. TENSAR® SS-1 and AR-1 geogrids were chosen to be used for this project. The SS-1 geogrid is a heat-sensitive material requiring insulation from a seal coat. The AR-1 geogrid has greater heat resistance capabilities. At the first installation site, the SS-1 geogrid was placed, nailed at one end, tensioned by using a pickup truck, a come-along, and a Dynamometer, and nailed at the far end and at intermediate points to secure the geogrid. Following placement of the grid, the contractor placed two, 57-mm (2-1/4-in.) lifts of previously milled asphaltic concrete materials. The material was characterized as being in a very tight condition. A seal coat was placed and then placement of the asphaltic concrete surface was initiated. Large sections of the asphaltic concrete surface began coming up within two weeks after placement. All materials within this section were removed including the geogrid, rotomilled material, seal coat, and surface layer. These materials were replaced with 114-mm (4-1/2-in.) asphaltic concrete base, TENSAR® AR-1 geogrid, and an asphaltic concrete surface course. Researchers did not believe that the SS-1 geogrid contributed to the failed section but attributed the failure to the rotomilled materials. The AR-1 was placed in accordance with the manufacturers directions. However, in some areas the geogrid appeared to ride up toward the top of the asphaltic concrete surface. In these areas, the asphalt over the fault was removed, the geogrid cut and removed, and the remaining geogrid nailed back in place. These areas were overlaid and have apparently performed well for more than three years.

The Maine Department of Transportation's Research and Development Section have been evaluating the use of geogrids for reinforcement of asphaltic concrete pavements since the late 1970's, [27]. A 1984 project called for the use of a TENSAR® geogrid and a Bates Terra-firma to alleviate a severely distressed pavement condition. The pavement was located on poor foundation materials, was severely rutted, and had poor drainage characteristics. Construction difficulties included development of a hump or wave in the geogrid and that the geogrid was not in complete contact with the underlying pavement. However, a 1988 report indicated the test sections were in good condition.

The Minnesota Department of Transportation utilized TENSAR® AR-1 geogrid on a pavement rehabilitation project in Wright County in 1984, [27]. The condition of the pavement was considered to be good, although it was heavily cracked. The AR-1 geogrid was placed on the existing asphaltic concrete surface, tensioned and nailed in place. A chip seal of asphalt emulsion and pea gravel was placed over the geogrid. Traffic was allowed on the section overnight which caused most of the chip to be tracked off by traffic, however there were no problems experienced during paving operations the following day. Wright

County engineers did not considered the work satisfactory and planned additional work during the Spring of 1985.

The New Mexico State Highway and Transportation Department reported on the use of TENSAR® AR-1 geogrid on two projects. One project was constructed in October 1985. The newly constructed pavement section existed of 152 mm (6 in.) of untreated base overlain by 152 mm (6 in.) of plant mix asphalt. Inspections conducted in 1990 detected distresses and it was reported the TENSAR® AR-1 geogrid did not prevent pavement cracking. The second project involved the use of the TENSAR® AR-1 geogrid in the rehabilitation of an off ramp of Interstate 40 near Santa Rosa. The project manager considered the July 1988 installation a success.

The use of high-strength, polypropylene grid can be regarded as one way to reinforce an asphaltic concrete pavement to diminish the distressing effects of traffic such as rutting and shoving. After more than a decade of laboratory and field testing, the key to using these high-strength materials for reinforcement efficiently lies in the ability of the contractor to suitably install the grid such that it can provide added tension to the asphaltic concrete pavement.

Polymer-Modified Asphalts

The advantage of using a polymer-modified asphalt is that it gives more extendibility and elasticity to the pavement. Therefore, a polymer-modified asphaltic concrete pavement is more resistant to rutting, low temperature cracking, and fatigue cracking than is a conventional asphaltic concrete. Asphalt is a combination of many hydrocarbon materials which is produced mainly through the distillation of petroleum crude oil. Polymer-modified asphalt is the combination of polymers and asphalt which is intended to improve the quality of the asphalt binder. There have been many laboratory studies that have shown that polymer-modified asphalt can increase rutting resistance, enhance low-temperature cracking, improve the adhesive friction of the asphaltic concrete surface, provide better adhesion and cohesion of the aggregate binder, and increase the strength of the pavement. However, there are many differences between the laboratory and actual field conditions. For instance, the cooling of the asphalt in the lab is more rapid than in the field [28]. When polymer-modified asphalts were introduced, they were really promoted to reduce or eliminate low-temperature cracking. However, it has since been determined that low-temperature cracking resistance is controlled by the base asphalt and not the polymer.

Polymers basically work by decreasing the point at which the asphalt will fracture due to extreme cold and increase the flow temperature due to extreme heat. Elastomeric polymers

(natural, latex and styrene rubber) are used to lower the fracture point from about 0°C to -23°C (32°F to -10°F) to prevent cold weather cracking. Conversely, plastomeric polymers (ethylene and acrylic copolymers) raise the softening point of an asphalt.

In the field, polymers in asphalt have made such great advances in highway paving that states such as New Mexico and Colorado have adopted polymer-modified pavements as the sole use for road repair. Other states also have experimented with polymer-modified asphalts for rehabilitation, [29 and 30]. The Tennessee Department of Transportation (TNDOT) has rehabilitated sections of I-40 and I-65 in the Nashville area with polymer-modified hot-mix asphalt. The average daily traffic on these interstates is approximately 40,000 vehicles per day and 11 percent trucks. The pavement thicknesses were 368 mm (14-1/2 in.) of asphaltic concrete over 254 mm (10 in.) crushed stone base. Apparently no binder course was used permitting the construction to proceed from the base course to the surface course, thus reducing labor costs. The main objective for the use of the polymer-modified asphaltic concrete was to relieve rutting in the pavement. Stapler report in the 1992-93 issue of *Asphalt*, that the project was a success as very little rutting had occurred after several seasons. Because the projects were so successful, TNDOT now specifies that all of Tennessee's interstate rehabilitation projects use polymer-modified asphalts.

Polymer-based asphalts appear to be a very cost effective way of adding extra durability to roadways. There are some fundamental problems associated with the use of polymers in asphalt however. Some polymers are incompatible with certain asphalts. For example, it was found that certain paraffinic asphalt types would dissolve the added polymer. The optimum particle size of the polymer is difficult to control. The particle size of the polymer will affect the viscosity of the asphalt binder. For a certain weight, the smaller the particle size, the larger the number of particles and therefore the higher the viscosity. Conversely, the larger the particle size, the lower the number of particles and the lower the viscosity. The low viscosity will increase the high temperature creep of the pavement under loading, [28]. It has been Kentucky's experience that hot-mix asphalt plants do not require modifications to produce a polymer-modified asphalt. Field operations also have to be investigated. Variations in standard paving techniques may be necessary for proper placement of polymer-modified asphaltic concrete pavement layers. Temperature of the polymer-modified asphalt must be consistently maintained between 149°C and 154°C (300°F and 310°F) during placement. The freshly laid mat must be rolled while still hot using vibrating roller having a high frequency and low amplitude, [31].

Kentucky has had varying success with the use of polymer modified asphalts. Fleckenstein reported on the use of Kraton® strenic block copolymers in an asphaltic concrete mixture placed on the KY15 Bypass in Perry County in a 1988 report, [32]. The purpose of the investigation was to determine whether the Kraton® asphalt additive would enhance

performance of the materials and prolong pavement life. Evaluations included construction inspections, performance inspections, and laboratory and field investigations. Construction of the 25-mm (1-in.) polymer-modified asphaltic concrete surface course was satisfactory with the exception of a low mat temperature around 93°C (200°F). Results of the assorted investigations varied. The polymer-modified asphaltic concrete surface was placed in areas that received the majority of the heavy-truck traffic. Rutting measurements indicated that the control section (containing a conventional Class I surface mix) had 2.4 percent more rutting within a five-month period after placement. Repeated load tests of field cores were performed. Test data of field cores indicated a higher resilient modulus and less potential for rutting in the control sections. If the control and modified mixtures were the same except for the differing asphalts, the modified asphalt, being stiffer, should have demonstrated higher resilient modulus values. It is likely that the degree of compaction differed for the two mixtures. Skid resistance tests indicated a slightly higher skid number for the control sections, 36 versus 34 for the Kraton® sections. Because of the conflicting data, no conclusion was made as to the whether the Kraton® asphalt additive enhanced performance or prolong pavement life.

However, the use of polymer-modified asphaltic concrete mixtures proliferated throughout the state with little assessment of its performance. During 1988-89, the UKTC investigated several asphalt modifiers in laboratory and field studies, [33]. Placement of five modified asphalt pavement sections and a control section on KY 15 in Perry County was uneventful. The asphaltic concrete modifiers included a polymer, a polyester fiber, a polypropylene fiber, and Vestoplast, a European modified asphalt system. The fifth section was designated as a Class N mixture which was a coarser mix than the Class A mix. Samples of the mixtures were compacted in a temporary laboratory set up at the hot mix plant specifically for sampling purposes. The laboratory evaluation program included Marshall stability, resilient modulus, moisture damage, tensile strength, and freeze-thaw susceptibility. It was determined that the polymerized and Vestoplast mixes were more desirable than the other mixes with respect to resistance to rutting. There were no significant improvements from the two fibrous mixes. The rutting susceptibility of the fibrous mixtures was greater than other mixtures. However, it was thought that fatigue and thermal cracking may be retarded in these sections. Although the field sections had not been in service for sufficient time to provide conclusive evidence as to modifiers reducing the susceptibility of asphaltic concrete pavements to rutting, shoving, fatigue cracking, and thermal cracking, Mahboub concluded that modifiers tend to remedy one mode of distress while having no effect, or possibly undesirable effects, on other modes of distress, [33]. The optimum design appears one that compromised between different modes of pavement distress, traffic load, and environmental factors.

Another extensive laboratory evaluation and field trial followed wherein several modified asphalt mixture systems were placed on KY 80 in Pulaski County in August 1990, [34]. These systems included Vestoplast, polypropylene and polyester fibers, Gilsonite, and two polymers, identified only as PMAC#1 and PMAC#2. Laboratory testing included Marshall stability and flow, mixture air voids and density, indirect tensile strength, susceptibility to moisture damage and freeze-thaw damage, resilient modulus, and repeated load deformation. Statistically-based comparative analyses were conducted to determine any significant differences in the performance potential of the different systems. The laboratory studies indicated that some of the modified systems had the potential to reduce premature pavement distresses. One of the polymer modified mixtures demonstrated the greatest potential for reducing pavement rutting while one of the fibrous mixtures demonstrated the greatest potential for reducing cracking. At the time the report was issued, sufficient long-term performance data had not been obtained to provide verification of the laboratory tests.

The two Kentucky trial projects, KY 15 in Perry County and KY 80 in Pulaski County, were thin overlays, typically 25 mm (1 in.), over existing material. Although there have been many thick-lift jobs of 100 mm (4-in.) or more in Kentucky that would be good comparisons, information on the construction and performance of these polymer-modified asphalt sections have not been reported upon.

Stone Matrix Asphalt

A group of highway professionals, including representatives from the Federal Highway Administration, the American Association of State Highway and Transportation Officials, Transportation Research Board, National Asphalt Paving Association and the Asphalt Institute, toured Europe during the Fall of 1990 to observe European highway technologies and contracting practices, [35]. One of the most interesting aspects of this tour was the introduction to stone matrix, or stone mastic, asphalt (SMA) pavement. SMA was first used as a heavy-duty paving mixture by German contractors in the 1960's. SMA provided a rut-resistant pavement that extended the service life of the pavement as much as 25 to 50 percent, [36]. SMA is a gap-graded asphaltic concrete mixture that includes a high asphalt content, a high dust content, and a stabilizer. The typical aggregate gradation for a SMA mixture follows the 30-20-10 rule of thumb. That is, the typical 13-mm (½-in.) mix contains no more than 30 percent passing the 4.75 mm (No. 4) sieve, 20 percent passing the 2.36 mm (No. 8) sieve, and 10 percent passing the 75 μ m (No. 200) sieve. By contrast, most conventional hot mix asphalt mixtures contain aggregate of a uniform gradation with up to 60 percent passing the 4.75 mm (No. 4) sieve and six percent passing the 75 μ m (No. 200) sieve. SMA mixtures have a high asphalt content typically ranging from 6.0 to 7.5 percent. Fibers, either cellulose or mineral wool, are used to bind up the extra asphalt and keep it

from draining out of the mix as it is being handled. The asphalt content is designed using a voids analysis procedures of the Marshall Mix Design Method. Samples are prepared using 50 blows per side. The target air voids for a typical SMA mixture is three percent.

Following the European tour, five states placed experimental SMA pavements during the Summer of 1991. The demonstration projects resulted from close cooperation of the state DOT, contractor, FHWA, and European technical support. Initial demonstration projects were constructed in Wisconsin, Michigan, Georgia, Missouri, and Indiana. By the end of 1992, Alaska, Maryland, Ohio, Illinois, Texas, and Virginia had constructed experimental projects. In addition to placement of the experimental SMA mixes, conventional, dense-graded mixtures were placed for performance comparisons. There were no reported difficulties encountered during the experimental placements. All operations stressed the need for early compaction. Roller operators were encouraged to compact as close to the paver as possible. The experimental projects showed that the costs of SMA mixes are estimated to be only 15 to 30 percent higher than conventional hot-mix asphalt mixtures. Performance of these experimental sections is being monitored by the respective states and compared to their conventional hot-mix asphalt paving mixtures. However, performance of these initial test sections has not been reported upon. All reports resulting from these research activities have detailed only the laboratory mixture design procedures for the SMA. Conversations with Department of Transportation personnel in Wisconsin and Michigan, where the first projects were constructed, indicate there are no significant difference in the experimental and control sections but also that the evaluation period for some of the experimental sections had not been long enough for a distinct performance difference to develop. The SMA mixes have been placed in a variety of loading and environmental situations and the experimental sections will provide valuable information as they begin to age and accumulate loads.

Brown, in a 1993 report, detailed laboratory evaluations performed at the National Center for Asphalt Technology on SMA materials from the 1991 Michigan demonstration project, [37]. The objective of the laboratory study was to evaluate the sensitivity of SMA mixture properties to changes in proportions of various mixture components. Samples were prepared for 17 different mixture variations. Samples were prepared in the U.S. Army Corps of Engineers gyratory testing machine. The machine was set to produce a density equivalent to 50 blows with the Marshall hammer. Dense-graded hot-mix asphalt specimens were prepared for comparison testing. The study was somewhat limited in that only one aggregate, one asphalt, and one additive were used to produce both SMA and dense-graded hot-mix asphalt mixtures. Findings reported by Brown were that the hot-mix asphalt mixtures performed better in many laboratory tests than did some SMA mixtures having variations in the job-mix formula. These results pointed out the need for close control of the aggregate gradation to insure best performance. SMA mixtures often

performed best in the gyratory shear test and the confined creep tests. These two tests are indicators of rutting resistance and will be most useful to evaluate the quality of future SMA mixes.

Large Stone Mixtures

Pavement failures due to improper design procedures, poor construction, and/or poor materials are not surprising; however, failures detected and not attributable to these causes are, [38]. The origin of failures in pavements having been properly designed and constructed prompted Davis and other researchers to reconsider and further investigate the basic principles of pavement design, [39, 40, 41, 42, and 43]. The type of failure most often seen was pavement rutting. Anani defined rutting to be the formation of twin longitudinal depressions under the wheel paths from a progressive accumulation of permanent deformation in one or more pavement layers, [40]. A global increase in the frequency of this problem began on a large scale in the mid to late 1970's. Initially, the increase in pavement rutting appeared to be the result of stripping. This initial conclusion was premised on the considerable alterations in crude oil refinement caused by the Arab Oil Embargo. However, additional research and data collection revealed that older pavements were also showing similar distress patterns. Subsequently, it was observed that the rate and magnitude of rutting was a function of both internal and external factors. Specifically, the internal factors include binder properties, aggregate, mix, and thickness of the pavement layers. The external factors include load and volume of traffic, tire pressure, temperature, and construction practices.

Data collected from trucking companies during this same time period suggested that the tire pressure for almost all trucks had increased to approximately 1,034 kPa (150 psi). Further study of failed sections convinced Davis that this was the most probable explanation for the sudden increase in pavement failure, [38]. The pavement designs were derived from AASHTO specifications that had considered tire pressures no higher than 483 kPa (70 psi) and failures at the 1,034 kPa level could be expected. An increase in gross weights and contact pressures were also cited as reasons for increased pavement failures. For example, in Kentucky the legal maximum weight limit on certain coal-haul corridors is 54,430 kg (120,000 lbs). However, it was observed that truck weights of 68,038 kg (150,000 lbs) to 81,646 kg (180,000 lbs) were not uncommon, [44].

It was ultimately realized by pavement researchers that the sudden change in external factors was the controlling factor and indicated shearing failure rather than stripping failure. Davis stated that as aggregate slides over aggregate in shear failure, the asphalt film would be scrubbed from the sliding surfaces, [38]. Davis hypothesized the pavement

would dilate during this type of failure and the enlarged pores would allow surface water to penetrate the pavement revealing a condition that resembled stripping. Davis concluded that lowering tire pressures would stabilize in-place pavements, but an increase in shear strength could enhance future designs, [38]. Two ideas were advanced to increase shear strength: 1) increase cohesion of the mix, 2) increase the top size of the aggregate in relation to volume concentration and pavement thickness. Although the former had been tested successfully in New York by the Koppers Company, the latter has been the focus of most recent research. The reason for the focus on increasing the top size of the aggregate is the cost effectiveness of altering the aggregate size compared with developing more efficient binders.

The Kentucky Transportation Cabinet (KTC) and the University of Kentucky Transportation Center (UKTC) have been pioneers in the development of large-stone bituminous mixes for heavy duty pavements. KTC and UKTC developed a large-stone base course mix, called Class K Base, containing 51-mm (2.0-in.) maximum size aggregate for heavy-duty pavements along Kentucky's coal-haul corridors. The mixture is designed using 95 mm (3-¾ in.) by 152 mm (6 in.) Marshall specimens compacted 112 blows each side. Requirements for stability, flow, and voids in the mineral aggregate (VMA) are 1,815 kg (4,000 lbs) minimum, 6.1 mm (0.24 in.) maximum, and 11.0 percent minimum, respectively. Air voids of the compacted mixture are maintained between 3.5 and 6.5 percent. Retained tensile strength (RTS) of the compacted mixture must be at least 65.0 percent. Target densities after compaction of the large-stone mixture are 92 to 96 percent of the solid density calculated from the maximum specific gravity of the plant-mixed material.

Laboratory testing has shown that the Class K, or large-stone mixture, is a much improved design when compared to Kentucky's standard Class I bituminous concrete base. Performance-oriented laboratory testing indicated higher levels of structural capacity and rutting resistance for the large-stone mix when compared to the standard mix, [39]. Specimens were evaluated relative to resilient modulus, and static and dynamic creep. Mahboub and Allen described modulus of elasticity as a parameter that relates the forces causing deformation to the actual deformation. Higher moduli are interpreted to indicate more resistance to deformation and deflection; therefore, a longer pavement life, [39]. Laboratory data indicate approximate resilient moduli values of 6,895 MPa at 24.4°C (1,000,000 psi at 76°F) and 1,379 MPa at 40°C (200,000 psi at 104°F) for the Class K specimens. Values for the conventional Class I base were significantly lower. Resilient moduli values for Class I specimens were 1,724 MPa at 24.4°C (250,000 psi at 76°F) and 345 MPa at 40.0°C (50,000 psi at 104°F). The data indicate a higher level of structural capacity of the Class K base mix as compared to the Class I base mix.

A mechanistic technique was also used to characterize pavement deformation. Static and dynamic (cyclic loading) techniques were employed by Mahboub and Allen, [42]. The static creep test consisted of loading a sample for a set period of time (one hour) under a constant load (200 kPa or 29 psi). The dynamic creep test was conducted for cyclic loading applied at a frequency of 1.0 Hz. Laboratory data reported by Mahboub and Allen show the Class K base mix to be less susceptible to permanent deformation than the conventional Class I base mix, [42]. Mahboub and Allen believed the stone to stone contact afforded by the large-stone mixture reduced the probability of plastic flow owing to low air voids and/or higher densities.

Anderson described concerns and potential problems during the planning stages of the Class K base, [44]. There were initial concerns that the large-stone mixture would cause excessive plant damage, as well as, excessive auger wear on pavers. However, no unusual problems were encountered during initial placements, [45]. Achieving a complete, uniform coating on the large aggregates was a problem using conventional mixing times. However, this problem was alleviated simply by increasing the mix time. The increased mix time did not significantly affect operations. Due to the range in sizes, Class K base was considered susceptible to segregation. Special attention had to be given to stockpiling techniques, placement of large aggregate in the cold feed bins, discharge of the mix from the silo into the haul trucks, delivery from the truck to the paver, elimination of the paver dumping the hopper between truckloads, and retention of sufficient mix above the conveyor. Adequate compaction had to be maintained to gain maximum benefits from the large-stone mixture. Special test strips were constructed prior to actual placement and stringent compaction testing was performed. Anderson reported that Class K base with large aggregate was no more difficult to compact than Class I mix, [44]. The Class K base on the Louisa Bypass project was placed in three 102-mm (4-in.) lifts, [45]. Although not a problem in the initial placements, aggregate fracturing during compaction was thought to be a potential problem. The Louisa Bypass project is still being monitored for performance.

Researchers appeared to agree unanimously as to the benefits of large-stone bituminous mixes for heavy-duty pavements. It was reported that the high volume concentration of aggregate with the top size of the aggregate $\frac{2}{3}$ the thickness of the pavement layer would increase the shear strength of the pavement layer to sufficiently withstand increased tire pressures and wheel loads, [44].

Summary

Methods most commonly used to enhance asphaltic concrete pavement performance at intersections have been whitetopping, geogrid reinforcement, and use of polymer-modified

asphalts. Other promising techniques are stone-mastic asphaltic concrete and large-stone mixtures. The literature review indicated that the excellent performance of concrete overlays of asphaltic concrete pavements was attributable to the fact that the concrete overlay makes use of the existing flexible pavement for support. Using an existing flexible pavement for support strengthens the overall pavement structure. The bond developed between the asphaltic concrete and the Portland cement concrete enables the asphaltic concrete layer to take a significantly higher proportion of the stresses, thereby reducing the overall stresses in the concrete layer. Minimum slab thicknesses are based on expected traffic levels and underlying support values of the existing pavement structure. The overall safety of the pavement surface is increased by eliminating rutting of the pavement surface.

The concept of tensile reinforcement of asphaltic concrete layers has existed for many years and has been attempted with various materials with varying success. The idea is to reinforce the flexible pavement to increase tensile strength and extend the life of the pavement. The use of high-strength, polypropylene grid may be regarded as one way to reinforce an asphaltic concrete pavement to diminish the distressing effects of traffic such as rutting and shoving. After more than a decade of laboratory and field testing, apparently the key to using these high-strength materials efficiently for pavement reinforcement lies in the ability of the contractor to suitably install the grid such that it can provide added tensile strength to the asphaltic concrete.

The advantage of using a polymer-modified asphalt is that it gives more extendibility and elasticity to the pavement. Therefore, a polymer-modified asphaltic concrete pavement is more resistant to rutting, low temperature cracking, and fatigue cracking than is a conventional asphaltic concrete. Polymer-modified asphalt is a combination of polymers and asphalt which is intended to improve the quality of the asphalt binder. There have been many laboratory studies that have shown that polymer-modified asphalt can increase rutting resistance, enhance low-temperature cracking, improve the adhesive friction of the asphaltic concrete surface, provide better adhesion and cohesion of the aggregate binder, and increase the strength of the pavement. In the field, polymer-modified asphaltic concrete has made such great advances in highway paving that states such as New Mexico and Colorado have adopted polymer-modified asphalt as the sole pavement repair material. Tennessee specifies polymer-modified asphalt for all interstate rehabilitation projects.

Polymer-based asphalts appear to be a very cost effective way of adding extra durability to roadways. There are some fundamental problems associated with the use of polymers in asphalt as some polymers are incompatible with certain asphalts. For example, it was determined that certain paraffinic asphalt types would dissolve the added polymer. Also, the optimum particle size of a polymer is difficult to control. The particle size of the polymer

affects the viscosity of the asphalt binder. Low viscosity will increase the high temperature creep of the pavement under loading. Additionally, hot-mix asphalt plants must be modified to produce a polymer-modified asphalt. Field operations also have to be changed from those associated with standard paving techniques. Lay-down temperature, speed of the paving auger, and rolling techniques all play an important role in the success of a polymer-modified asphalt pavement. Kentucky has several projects wherein polymer-modified asphalts were used to reduce the rutting potential of the asphalt wearing surface. However, sufficient documentation of their field performance is not available.

Stone mastic asphalt (SMA) is a gap-graded asphaltic concrete mixture that includes a high asphalt content, a high dust content, and a stabilizer. SMA is a heavy-duty paving mixture that was first employed by German contractors in the 1960's. Since its introduction, SMA has provided a rut-resistant pavement that has extended pavement life by as much as 25 to 50 percent. The asphalt content is designed using a voids analysis procedure of the Marshall Mix Design Method. Target air voids for a typical SMA mixture is three percent. Numerous SMA mixtures have been placed experimentally in several states since 1991. There have been no reported difficulties associated with the placement of the SMA mixtures although the need for early compaction has been stressed at all operations. In addition to placement of the experimental mixes, conventional, dense-graded mixtures have been placed adjacent to the experimental SMA mixtures to facilitate direct comparison between the two pavement types. Thus far, performance comparisons have not been formally reported upon.

The Kentucky Transportation Cabinet (KTC) and the University of Kentucky Transportation Center (UKTC) have been pioneers in the development of large-stone bituminous mixes for heavy duty pavements. KTC and UKTC developed a large-stone base course mix, called Class K Base, containing 51-mm (2.0-in.) maximum size aggregate for heavy-duty pavements along coal-haul corridors. Testing has shown that this is a much improved design when compared to the standard Class I base relative to withstanding increased tire pressures and wheel loads of today's vehicles. Performance-oriented laboratory testing indicated higher levels of structural capacity and rutting resistance for the large-stone mix when compared to the standard mix. Initially, there were concerns that the large-stone mixture would cause some damage to the plant and paver. However, no unusual problems were encountered during initial placements. Increased mixing times allowed a complete, uniform coating on the large aggregates to be achieved. The increased mix time did not significantly affect operations. Due to the range in sizes, Class K Base was considered susceptible to segregation. Therefore, special attention had to be given to stockpiling techniques, placement of large aggregate in the cold feed bins, discharge of the mix from the silo into the haul trucks, delivery from the truck to the paver, elimination of the paver dumping the hopper between truckloads, and retention of sufficient mix above the conveyor. Adequate compaction had to be maintained to gain maximum benefits from the

large-stone mixture. Special test strips were constructed prior to actual placement and stringent compaction testing was performed. Field trials confirmed that the Class K base was no more difficult to compact than the conventional Class I base mix. Performance of the Class K base experiment located on the Louisa Bypass is still being monitored.

EXPERIMENTAL SITES

Task B of the work plan for this study involved identifying a limited number of prematurely distressed pavements within the 12 highway districts across Kentucky. The method used to accomplish this task involved distributing a questionnaire to the highway Districts requesting that their Operations Engineers nominate two or three intersections or approaches that demonstrated significant premature pavement distress-related problems, such as deep rutting and severe washboarding. The questionnaire also requested Operations Engineers to identify one or two intersections or approaches that, historically, had not exhibited excessive pavement distresses early in the design life of the pavement although they were of similar design and construction and carried similar traffic volumes and loadings.

Ten of the 12 KYDOH highway districts responded to the questionnaire. UKTC personnel performed condition surveys at sites nominated for study by Operations Engineers in the responding districts. The investigations included performing visual condition surveys of the sites; photo-documenting each site; determining depth of rutting in each wheel path; and, observing the length from the intersection and severity of washboarding. Attempts were made to secure construction plans for each site. The original pavement thicknesses and material designs were determined from the as-built plans. Traffic histories for each site were obtained from the Division of Planning. It was intended that this information be used to rank the nominated sites and to choose the most severely distressed intersection pavements for further investigation.

KYDOH had already initiated several projects to rehabilitate some sites where the flexible pavement had become maintenance problem. In essence, KYDOH was already proceeding with rehabilitation at several sites before this study actually began. The research focus shifted to documentation of the different rehabilitation and/or replacement techniques being employed by KYDOH and performing follow-up inspections to determine the practicality of the rehabilitation technique. Those experimental sites for this study are described herein.

US 31E & US 68/KY 80 -- Barren County

The intersection of US 31E and US 68/80 was reconstructed as part of the US 31E widening project at the city of Glasgow in Barren County. Prior to beginning reconstruction in July 1990, rutting of the bituminous concrete pavement at the intersection was up to 51 mm (2 in.) on US 31E with severe shoving up to 15.2 m (50 ft) from the intersection. US 68/80 had rutting up to 19 mm (0.75 in.). Traffic counts in June 1990 indicated 26,709 vehicles per day use the intersection of which approximately nine percent is heavy truck traffic.

US 68/80 intersects US 31E at approximate Station 899+86 of US 31E. Mainline reconstruction of US 31E included adding lanes and resurfacing the existing lanes. Existing pavement within the experimental intersection was removed and reconstructed with PCC pavement. Mainline pavement of US 31E outside the PCC intersection was 229 mm (9 in.) of Class I bituminous concrete. The rigid pavement began at Station 894+75 (US 31E) and ended at Station 902+85. From the centerline of US 31E, the PCC pavement extended 45.7 m (150 ft) on the east leg of US 68/80 and 61.0 m (200 ft) on the west leg.

Pavement design of the intersection was 229 mm (9 in.) of PCC on 127 mm (5 in.) of untreated drainage blanket on 216 mm (8.5 in.) of DGA on a subgrade reinforced with high density plastic geogrid TENSAR® BX-100. Pavement joints were spaced at 9.3 m (15 ft.) intervals. Construction proceeded in phases with minimal closures to traffic. The PCC plant mix was 340 kg (750 lbs) of Type 1 cement, 739 kg (1,630 lbs) of coarse limestone aggregate, 630 kg (1,390 lbs) of fine aggregate, and approximately 86 kg (190 lbs) of water. Plasticizer and air entrainment admixtures were used as needed to improve characteristics of the fresh concrete mixture..

The Special Note For Portland Cement Concrete Pavement/24/48/72 (Experimental) (Appendix A) permitted alternate PCC mixtures with the requirement that 2.41 MPa (3,500 psi) compressive strength be achieved prior to opening to traffic. The mixture used was the 48-hour (PCCP/48) design strength.

UKTC Material Property Tests

The Special Note For Portland Cement Concrete Pavement/24/48/72 (Experimental) permitted a slump of 51 to 178 mm (2 to 7 in.) and air content of 6.5% ± 1.5%. Several truck loads were rejected on the first day of PCC placement because of low air content. Adjustments to batch size and water content alleviated the low air content problem. Monitoring of the fresh concrete at the site by KTC personnel indicated slumps ranging from 102 to 203 mm (4 to 8 in.) and averaging 147 mm (5.8 in.). Air contents ranged from 4.5 to 9.0% and averaged 6.4%. Results of field tests are shown in Table 1.

Table 1. Material properties for on-site tests of PCC conducted by UKTC.

DATE	ACCEPTED SLUMP		ACCEPTED AIR CONTENT	REFUSED AIR CONTENT
	(mm)	(in.)	(%)	(%)
10/8/91	114	4.5	7.2	4.0
10/8/91			7.0	3.0
10/8/91			4.5	3.8
10/8/91	173	6.8	6.2	
10/8/91	203	8.0	7.0	
10/8/91	114	4.5	5.1	
10/11/91	127	5.0	6.2	
10/11/91	140	5.5	7.8	
10/11/91	152	6.0	6.7	
10/11/91	165	6.5	9.0	
10/11/91	178	7.0	8.0	
10/11/91			5.0	
10/15/91	152	6.0	5.5	
10/15/91			4.5	
10/15/91	102	4.0		

UKTC personnel cast eight sets of specimens, each set consisting of four cylinders and two prisms, (ASTM Test Method) for laboratory tests. Compressive strength was determined at ages ranging from 24 hours to 1,416 hours (59 days) with compressive strength at 59 days in the range of 4.14 MPa to 5.52 MPa (6,000 to 8,000 psi). Seven cylinders were tested at or near 48 hours with compressive strengths ranging from 1.98 MPa to 3.29 MPa (2,880 to 4,770 psi). Cylinders from the eighth set were not tested at 48 hours but two of them were tested at 25 hours and indicated compressive strengths of 2.26 and 2.43 MPa (3,280 and 3,520 psi). Five of the eight sets met the 24,132 kPa (3,500 psi) requirement at 48 hours. KTC compressive strength data are shown in Table 2.

Fourteen prisms cast from seven different PCC batches were tested by ASTM C-666, [46]. All prisms performed well with Durability Factors ranging from 97.0% to 99.5% and expansion ranging from -0.002% to 0.022%. Freeze/Thaw test results are included in Appendix B.

Table 2. Summary of UKTC laboratory tests of PCC cylinders.

SAMPLE NUMBER	DATE CAST	TEST AGE (hours)	COMPRESSIVE STRENGTH		UNIT WEIGHT	
			(Mpa)	(psi)	(kg/m ³)	(lb/ft ³)
G-1-A	10/08/91	27	1.83	2,650		
G-2-A	10/08/91	26	1.40	2,030		
G-3-A	10/08/91	24	1.68	2,430		
G-4-A	10/08/91	24	1.34	1,940		
G-1-B	10/08/91	48	2.40	3,480		
G-2-B	10/08/91	48	1.98	2,880		
G-3-B	10/08/91	48	3.31	4,800		
G-4-B	10/08/91	48	2.52	3,650		
G-1-C	10/08/91	168	3.29	4,770		
G-2-C	10/08/91	168	2.66	3,860		
G-3-C	10/08/91	168	4.27	6,190		
G-4-C	10/08/91	168	3.43	4,980		
G-2-D	10/08/91	1,416	3.59	5,210	2,256.7	140.9
G-3-D	10/08/91	1,416	5.56	8,070	2,319.2	144.8
G-4-D	10/08/91	1,416	4.31	6,250	2,274.4	142.0
G-A-1	10/11/91	27	1.63	2,360		
G-B-1	10/11/91	26	1.38	2,000		
G-C-1	10/11/91	24	1.31	1,900		
G-A-2	10/11/91	52	2.52	3,650		
G-B-2	10/11/91	52	2.32	3,360		
G-C-2	10/11/91	51	2.24	3,250		
G-A-3	10/11/91	168	3.30	4,780		
G-B-3	10/11/91	168	2.98	4,320		
G-C-3	10/11/91	168	2.80	4,060		
G-A-4	10/11/91	1,344	4.51	6,540	2,280.8	142.4
G-B-4	10/11/91	1,344	3.89	5,640	2,287.2	142.8
G-C-4	10/11/91	1,344	3.70	5,360	2,234.3	139.5
G-1	10/15/91	25	2.43	3,520		
G-2	10/15/91	25	2.26	3,280		
G-3	10/15/91	336	4.40	6,380	2,327.2	145.3
G-4	10/15/91	672	4.60	6,670	2,320.8	144.9

KYDOH Material Property Tests

KYDOH personnel cast 18 cylinders and obtained 18 cores of the pavement for compressive strength and PCC thickness determination. Four cores were taken from the PCC placed on October 15, 1991 and tested at 27 and 48 hours age. Compressive strengths at 27 hours were 2.41 MPa (3,500 psi) and 2.66 MPa (3,860 psi). Compressive strengths at 48 hours were 2.73 MPa (3,960 psi) and 3.01 MPa (4,360 psi). Cylinders cast from the same pour and field cured were tested at 27 hours and indicated compressive strengths of 2.32 MPa (3,360 psi) and 2.39 MPa (3,470 psi).

Sixteen cylinders from various PCC batches were cast by KYDOH personnel and tested at various ages up to seven days. Six of eight cylinders tested at 24-hours age did not achieve the target compressive strength of 2.41 MPa (3,500 psi). All cylinders tested at ages of two or more days exceeded 2.78 MPa (4,000 psi). All KYDOH cylinder data are shown in Table 3.

Table 3. Summary of on site material property and laboratory tests performed by KYDOH.

SAMPLE NUMBER	DATE CAST	TEST AGE (days)	COMPRESSIVE STRENGTH		SLUMP		AIR CONTENT (%)
			(MPa)	(psi)	(mm)	(in.)	
1	10/11/91	7	3.42	4,960	114	4.5	6.2
2	10/11/91	3	3.04	4,410	114	4.5	5.2
3	10/11/91	7	3.38	4,910	114	4.5	5.2
4	10/11/91	3	3.02	4,380	114	4.5	6.2
5	10/11/91	1	1.66	2,410	114	4.5	6.2
6	10/11/91	1	1.91	2,780	114	4.5	5.2
7	10/15/91	1	2.35	3,410	152	6.0	5.0
8	10/15/91	1	1.37	1,990			
9	10/15/91	1	2.88	4,180	173	6.8	5.8
10	10/15/91	1	3.00	4,350	152	6.0	5.0
11	10/15/91	7	3.67	5,320	152	6.0	5.0
12	10/15/91	7	3.99	5,780	173	6.8	5.8
13	10/16/91	1	1.97	2,860	102	4.0	7.0
14	10/16/91	7	3.68	5,340	102	4.0	7.0
15	10/16/91	2	2.79	4,040	102	4.0	7.0
16	11/07/91	1			76	3.0	5.2

Fourteen cores were obtained from locations throughout the intersection on January 24, 1992. Compressive strengths of the cores ranged from 2.44 MPa (3,540 psi) to 6.14 MPa (8,900 psi). The age of the PCC at testing was unknown. Depth of PCC as indicated by core length ranged from 226 to 269 mm (8.9 in. to 10.6 in.). All KYDOT core data are shown in Table 4.

Performance

Visual surveys of the intersection have been conducted annually. The visual surveys conducted in 1993 and 1994 revealed no significant distress, but a visual survey in June, 1995 indicated seven mid-slab transverse cracks and several expansion joints with missing sections of joint seal and raveling and cracking at those joints. The missing sections of joint seals are the construction related. Previous experience has shown that pavement joint faces must be thoroughly clean and dry prior to installing a silicone joint seal. Deterioration of the joint faces is likely due to weak concrete stemming from high water to cement ratios in the area of the joint. Based upon notes and observations made during construction, it is thought the mid-slab cracking has resulted from inadequate base compaction during construction. The mid-slab cracking observed in the seven slabs was minor and is not anticipated to have a detrimental impact on performance of the concrete pavement. A typical distressed expansion joint and mid-slab crack are shown in Figures 1 and 2.

Table 4. Compressive strength and depth of PCC as determined by PCC cores.

CORE NUMBER	COMPRESSIVE STRENGTH		DEPTH	
	(MPa)	(psi)	(mm)	(in.)
1	2.44	3,540	259	10.2
2	4.62	6,700	226	8.9
3	5.87	8,510	259	10.2
4			234	9.2
5	5.21	7,560	244	9.6
6	6.14	8,900	269	10.6
7	5.84	8,470	264	10.4
8	4.71	6,830	239	9.4
9	5.74	8,330	239	9.4
10	5.70	8,270	254	10.0
11	4.34	6,300	226	8.9
12	3.74	5,430	249	9.8
13	4.34	6,300	264	10.4
14	4.92	7,130	264	10.4



Figure 1. *Typical mid-slab crack of PCC pavement at intersection of US 31E and US 68/80*



Figure 2. *Typical distressed pavement joint at intersection of US 31E and US 68/KY 80.*

US 31E & KY 90 -- Barren County

During the reconstruction of US 31E at Glasgow, another problem intersection (KY 90 and US 31E) was selected for an experimental technique to address rutting and shoving. Prior to reconstruction, the bituminous concrete pavement had up to 51- mm (2-in.) ruts at the intersection. Approximately 24,500 vehicles passed through the intersection daily with 8.2 percent, or more than 2,000 of the total being, heavy trucks. Ky 90 intersects US 31E at Station 931+38 of US 31E. Mainline reconstruction of US 31E included adding lanes and resurfacing the existing lanes. Existing pavement at the experimental intersection was removed from Station 927+00 to Station 936+50 of US 31E and approximately 30.5 m (100.0 ft) from the centerline of US 31 E on each approach of KY 90. Pavement design within the intersection included a high density plastic reinforcing grid (TENSAR® BX 100) on the subgrade, 203 mm (8.5 in.) of dense graded aggregate, 102 mm (4.0 in.) of untreated drainage blanket, 229 mm (9.0 in.) of Class K base in three courses, and 32 mm (1.3 in.) of Class AK surface. A high density plastic reinforcement grid (TENSAR® AR-1) was placed under the top base course of 76 mm (3.0 in.). The Special Note for Bituminous Pavement Reinforcement is included in Appendix C in this report.

Construction

Reconstruction of the intersection began with the grade and base preparation of additional lanes and completion of the curb, gutter, and associated drainage facilities. The existing pavement structure and base were then removed to the redesigned grade. A roadbed reinforcement (TENSAR® BX 100) was placed on the subgrade and covered with 216 mm (8.5 in.) compacted DGA. The DGA was covered with 102 mm (4 in.) unbound drainage blanket (57 gradation). A Class K base of 229 mm (9 in.) was placed in three courses and was covered with 32 mm (1.25 in.) of Class AK surface.

The TENSAR® AR-1 was designed to be placed between the second and third course of Class K base or approximately 114 mm (4.25 in.) below the pavement surface. The TENSAR® AR-1 reinforced area was designed to extend from Station 927+00 to Station 936+50 on US 31E and from Station 48+90 to Station 51+00 on KY 90. The reinforcing was extended by means of a construction change order to approximately Station 54+00 on KY 90.

Removal of the existing pavement began on September 11, 1992 (Friday) at 6:00 PM. Work was confined to KY 90 from approximately Station 50+50 to Station 54+00 until that area was completed through the first base course and reopened to traffic September 12. The following weekend, September 18 and 19, the opposite side of the intersection, KY 90 from Station 48+90 to Station 49+50, was reconstructed in the same manner. KY 90 was opened

to traffic with only 76 mm (3.0 in.) base until US 31E was reconstructed and grade could be matched across the intersection. The entire intersection was then paved through the second base course.

During construction of the KY 90 westbound lanes, a small section of the subgrade reinforcing grid was omitted from the right turn radius onto KY 90 from southbound US 31E. After placement of the DGA, drainage blanket, and first 76-mm (3.0-in.) thick base course, traffic was permitted on the pavement until all US 31E was constructed to the same elevation. Within hours of being opened to traffic, the pavement section missing the subgrade reinforcement failed. Upon removal of that section of pavement, the reinforcing grid was placed and the pavement was reconstructed and has performed well.

When the entire intersection was reconstructed through the second base course, work began on the placement of TENSAR® AR-1 and completion of paving. Placement of the TENSAR® AR-1 began on October 1 on KY 90 east of US 31E (Station 50+50 to 54+00), and the widening lanes on US 31E northbound. TENSAR® AR-1 was placed to the change order limits on Ky 90 and design limits on US 31E right of the centerline of US 31E. Due to problems encountered during construction, TENSAR® AR-1 was deleted left of centerline of US 31E and on KY 90 (Station 48+90 to 50+50). A schematic of the reconstructed intersection indicating the area where TENSAR® AR-1 was used is shown in Figure 3.

Initially, the general construction procedure was to place TENSAR® AR-1 with the long axis parallel to centerline, secure end to end overlaps with "hog rings", spray TENSAR® AR-1 and Class K base with tack coat, tension and pin the TENSAR® AR-1 to the base, and overlay with the third course of Class K base. The TENSAR® AR-1 was tensioned by pinning one end to the base, affixing a clamp across the free end and to a pickup truck, pulling the TENSAR® AR-1, and pinning the TENSAR® AR-1 to the base (Figures 4 and 5). Pins used were 127-mm (5-in.) concrete nails with washers (Figure 6). The side lap, adjacent strings of TENSAR® AR-1, varied from 0 to 610 mm (0 to 24 in.).

Problems occurred first with construction traffic picking up the TENSAR® AR-1 due to the tack coat. The tack coat was deleted. Another problem occurred when the TENSAR® AR-1 developed rolls or waves between the paver and haul truck (Figure 7). The bituminous mixture would fall through and beneath the TENSAR® AR-1 or the TENSAR® AR-1 would fold. Large areas would either have reinforcement grid near the surface or folds in the grid. In some cases, the reinforcement grid could be seen at the surface of the final base course. Several attempts to increase TENSAR® AR-1 tension sufficiently to eliminate the development of rolls or waves were unsuccessful. The solution to rolls or waves in the TENSAR® AR-1 was to cut the material in 6.1-m (20.0-ft) sections and slide them into place (underlapping by 0.9 to 1.2 m (3.0 to 4.0 ft) the previous piece) as the paver progressed. The

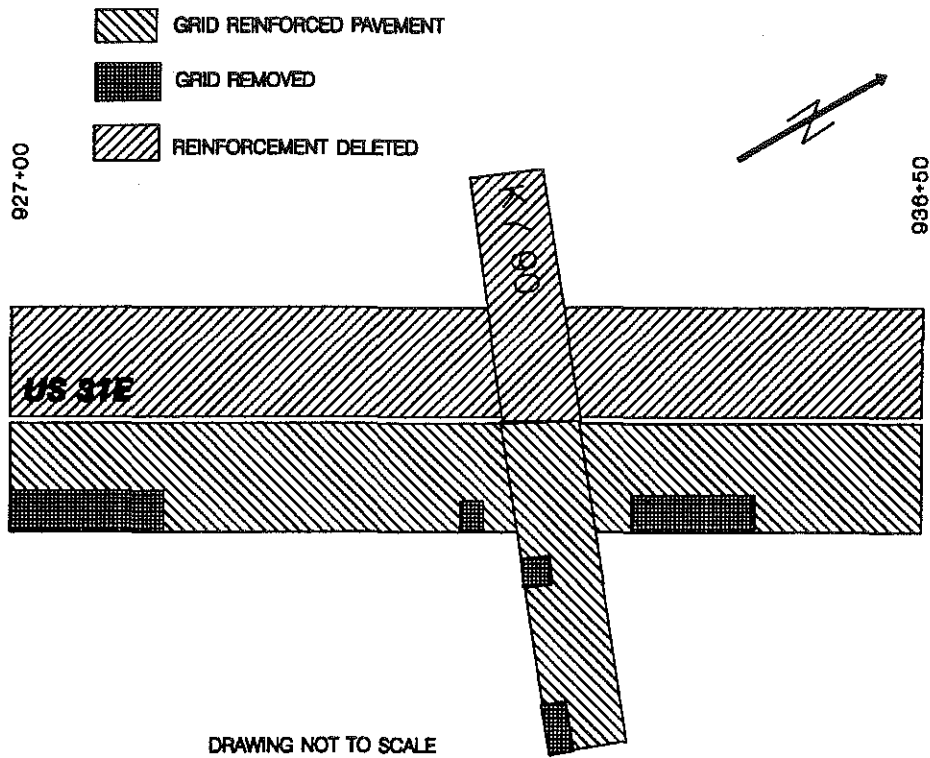


Figure 3. *Design and as-built location of TENSAR® pavement reinforcement at US 31E and KY 90.*

6.1-m (20.0-ft) sections were not fastened together, therefore, the reinforcement grid was not pretensioned where the cut sections were used.

After placement of the TENSAR® AR-1 and final base course had been completed right of centerline of US 31E, five areas of unstable or "spongy" pavement were located. The TENSAR® AR-1 was near the top of the base, not 76 mm (3 in.) below the top as shown in Figure 8, and appeared to have created a plane where interlocking of the reinforcement grid and aggregate did not develop. These areas were sawed and removed by pulling the TENSAR® AR-1 from the mat. The areas were backfilled with Class AK surface and the reinforcement grid was deleted. Those areas were from Station 927+00 to Station 928+30 (4.3-m or 14.0-ft width) and Station 932+32 to Station 933+70 (2.4-m or 8.0-ft width) on US 31E and three areas on KY 90. Total area removed and reinforcement grid deleted was approximately 460 m² (550 yd²).

Performance

Falling Weight Deflectometer (FWD) data were collected through the intersection in 1993. The data were analyzed to determine the effect of the reinforcement grid on pavement

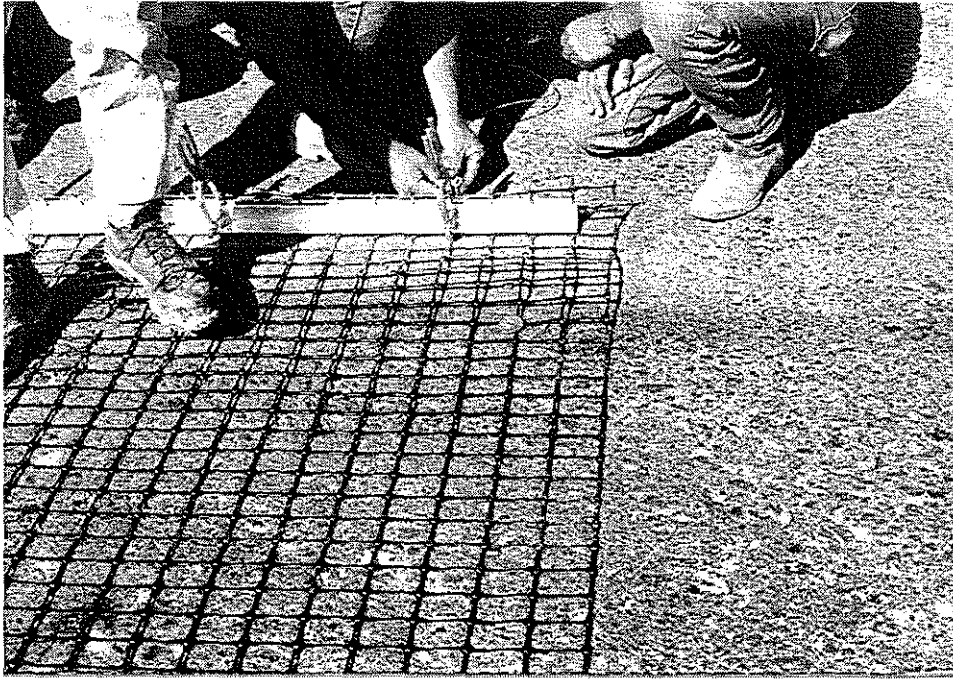


Figure 4. *Tensioning the TENSAR® reinforcing grid.*

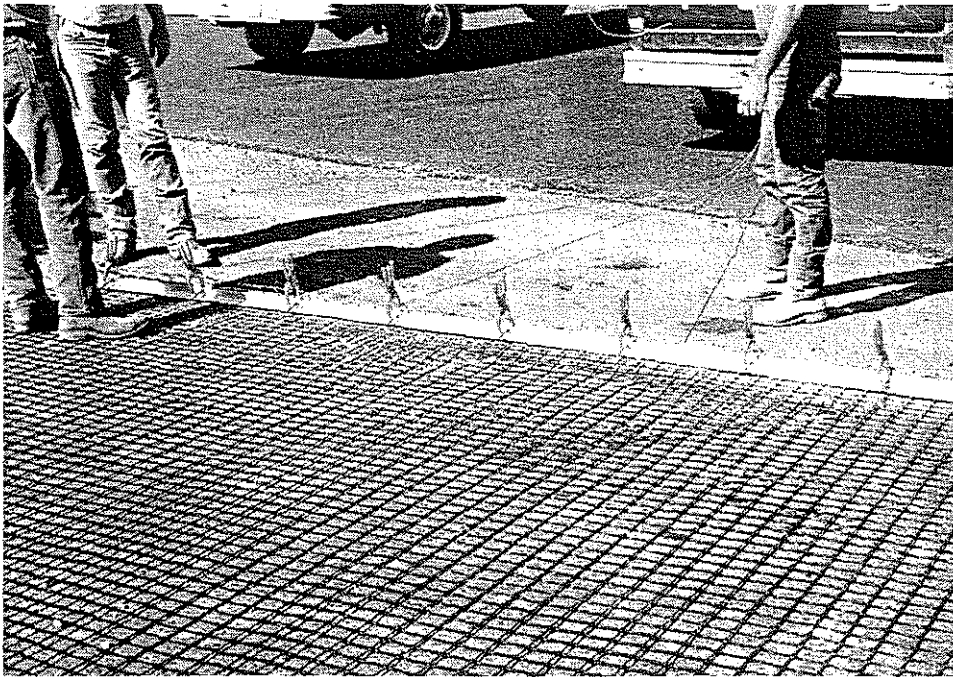


Figure 5. *Tensioning the TENSAR® reinforcing grid.*



Figure 6. *Pinning the reinforcing grid to the base course.*



Figure 7. *Waves developed between the paver and the haul truck.*

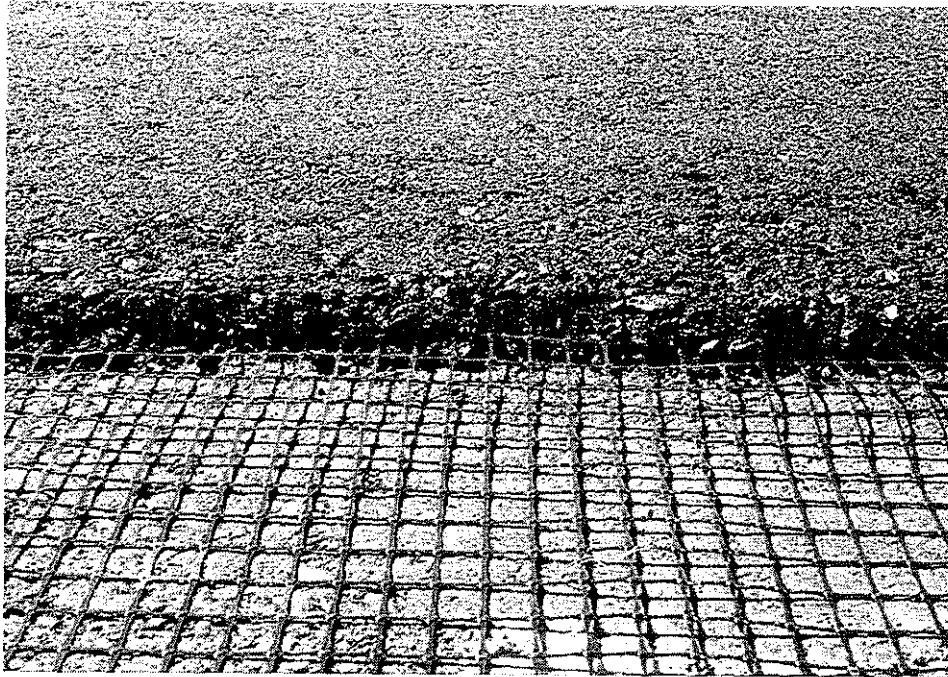


Figure 8. *TENSAR® reinforcing grid too near the top of the base course.*

response. Also, vertical and horizontal control points were established on the reconstructed surface to monitor potential shoving and/or rutting.

Pavement monitoring stations were established on each of the four legs of the intersection for the purpose of determining rutting and shoving. The stations were located perpendicular to centerline and approximately 6.1 to 12.2 m (20.0 to 40.0 ft) before the stop bar to avoid corner radii and skewed stop bars. Points were located on 0.3-m (1.0-ft) intervals beginning at the right curb for traffic approaching the intersection. Beginning points, end points, and intermediate points in the likely wheel paths were established by driving concrete nails into the pavement. Other points were marked with paint and reestablished when faded. A nearby benchmark was used for vertical control and the elevation of each point was determined by optical survey. The points were located on a string line between the beginning and end points so that shoving could be referenced to the initial string line.

Monitoring stations were established in November, 1992 immediately after the intersection surface was completed. Data were obtained five times in the succeeding 18 months. The change in elevation across the pavement ranged from 152 mm (0.5 ft) to more than 304 mm (1.0 ft), therefore simple cross sections do not adequately reveal rutting. Rutting was

determined by comparing cross section elevation to a computer generated string line which considers the high points across a lane or lanes as the base line. As of June, 1995, rutting of KY 90 eastbound was a maximum of 15.0 mm (0.6 in) in the right turn lane. Rutting had increased by 100% since 1994. Rutting of Ky 90 westbound was a maximum of 9.0 mm (0.4 in) in the right turn lane which was approximately a 25% increase since 1994. Rutting of US 31E northbound was a maximum of 14.0 mm (0.6 in) in a left turn lane with virtually no increase since 1994. Rutting of US 31E southbound was a maximum of 14.0 mm (0.6 in) in a through lane with approximately 10% increase since 1994 in most lanes but a 100% increase in the right turn lane. Rutting at the intersection is shown for 1994 and 1995 in Figures 9 through 12. Shoving or lateral displacement was 25.0 mm (1.0 in) on KY 90 which has a negative slope toward the intersection from both directions. Shoving on US 31E was approximately 15 mm (0.6 in.).

FWD tests were conducted at dynamic force levels of 5,443 and 6,804 kg (12,000 and 15,000 lb). Deflections of 0.406 mm (0.016 in.) or less were produced in the pavement. The effective Structural Number (SN) calculated from FWD data ranged from 5.5 to 7.0 with one outlying SN at 8.0. There is no significant difference in the average SN for southbound (non-reinforced) and northbound (reinforced) pavement if the outlying SN is not included. The calculated Structural Numbers for the intersection are shown in Figure 13.

US 62 & I-75 -- Scott County

Eastbound US 62 approaching the I-75 ramps on the east side of I-75 had been a severe problem for KYDOH maintenance personnel. In 1994, rutting at the intersection was reported by the maintenance engineer to be 152 to 203 mm (6.0 to 8.0 in.). The pavement was milled and replaced and within 4 months up to 102 mm (4.0 in.) of rutting had occurred. In October, 1994, eastbound US 62 from the traffic signal 82.3 m (270.0 ft) west toward the I-75 bridge was milled to a depth of 102 mm (4.0 in.) and replaced with an equal thickness of experimental PCC pavement. The PCC pavement extended across all eastbound lanes for a width of 14.6 m (48.0 ft). Details for the project are included in Appendix D of this report. The replacement pavement was a PCCP/24 design that required compressive strength of 2.41 MPa (3,500 psi) in 24 hours. Synthetic fibers were added to the PCC at a dosage rate of 1.7 kg/m³ (3.0 lb/yd³) of concrete. Traffic loops were installed prior to placement of the PCC pavement. All joints (longitudinal and transverse) were on 1.8 m (6 ft) intervals, and were sealed with silicone sealant. Unfortunately, UKTC personnel did not observe construction of the PCCP inlay. However, KYDOH inspection personnel reported a very workable mixture with no construction difficulties encountered. Special Notes for the experimental PCC are included in Appendix D of this report.

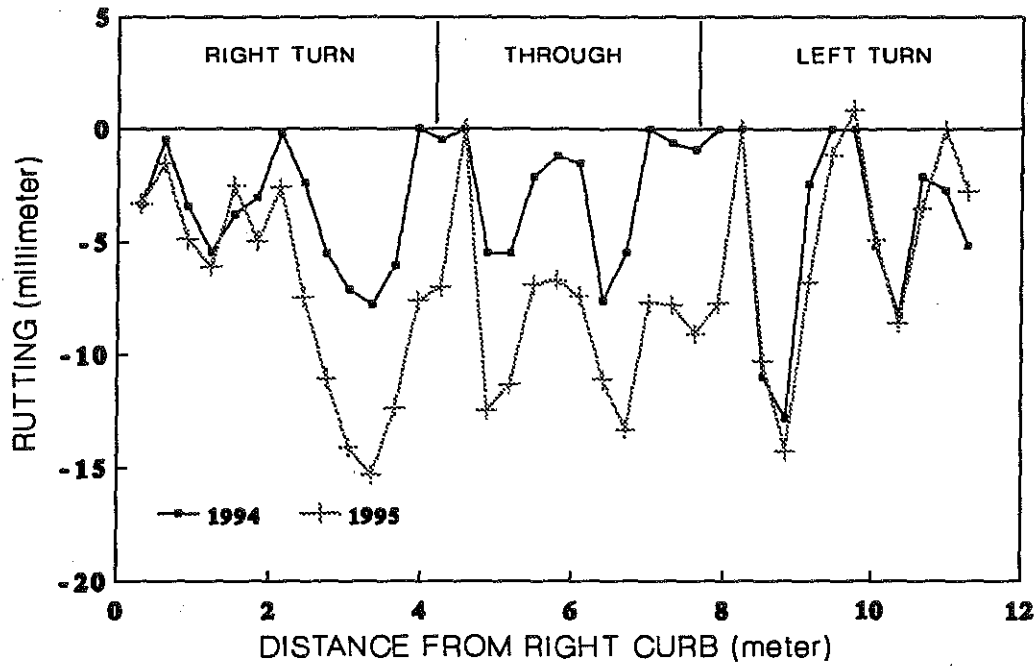


Figure 9. Rutting of KY 90 east bound at US 31E.

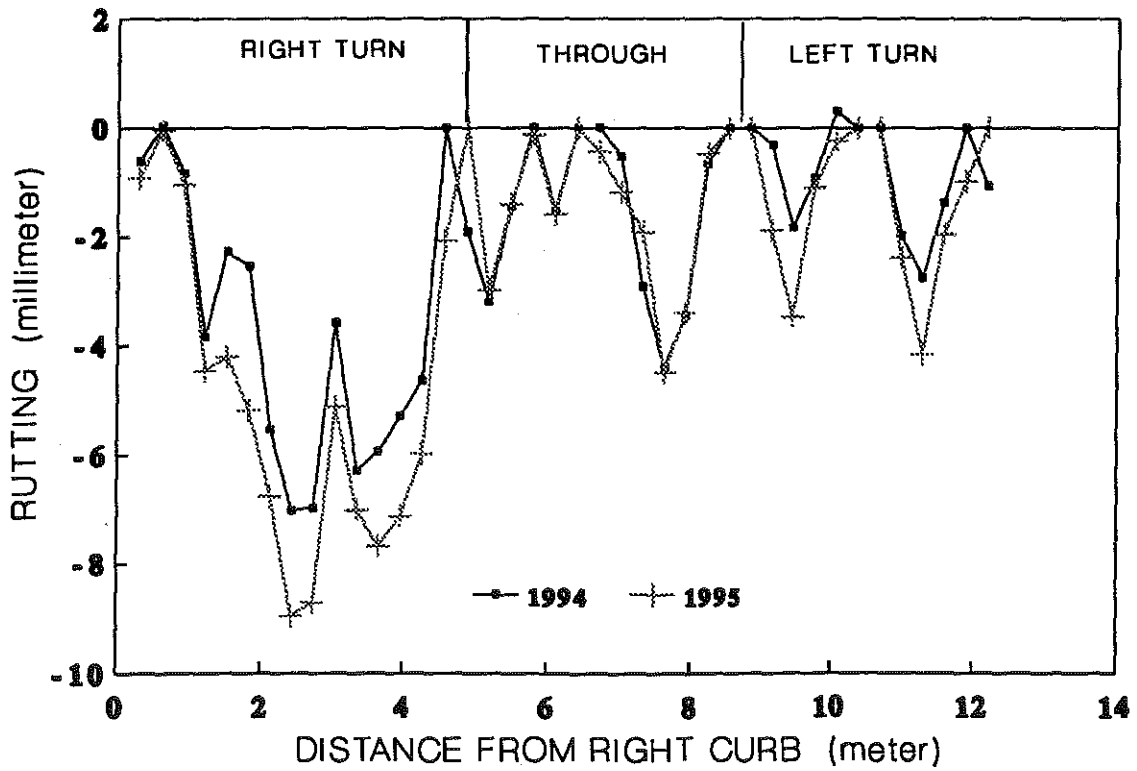


Figure 10. Rutting of KY 90 west bound at US 31E.

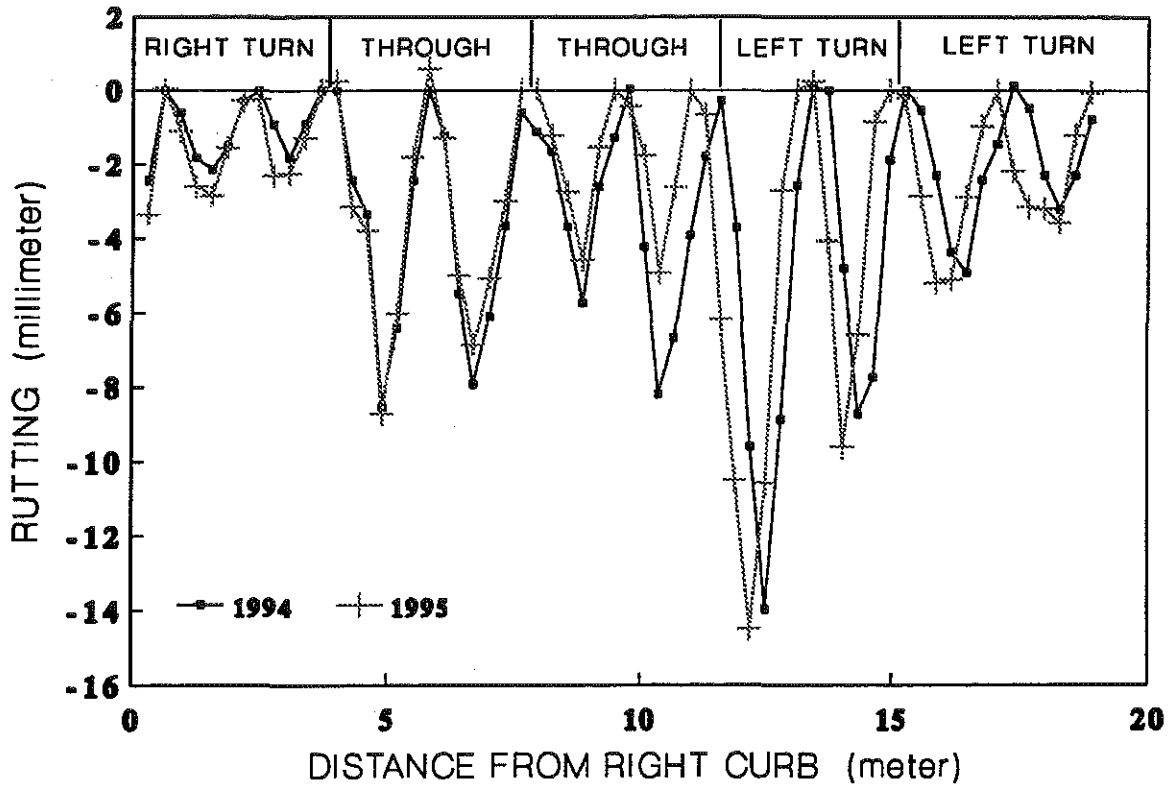


Figure 11. Rutting of US 31E north bound at KY 90.

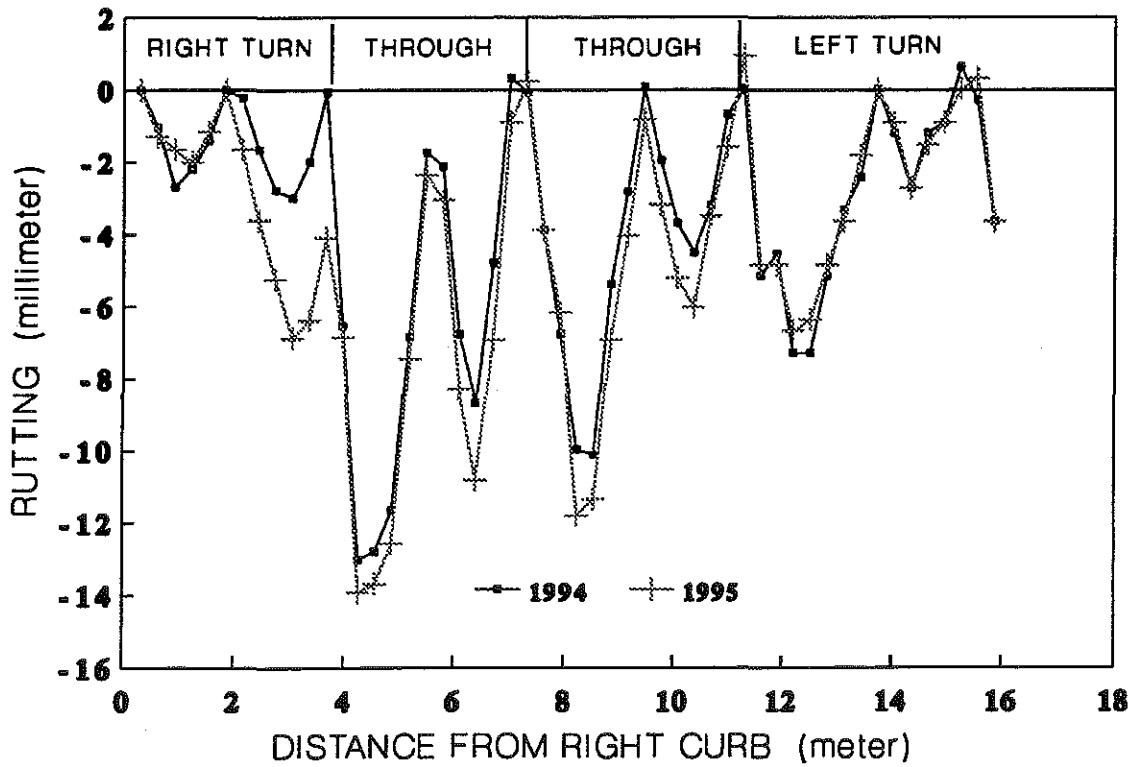


Figure 12. Rutting of US 31E south bound at KY 90.

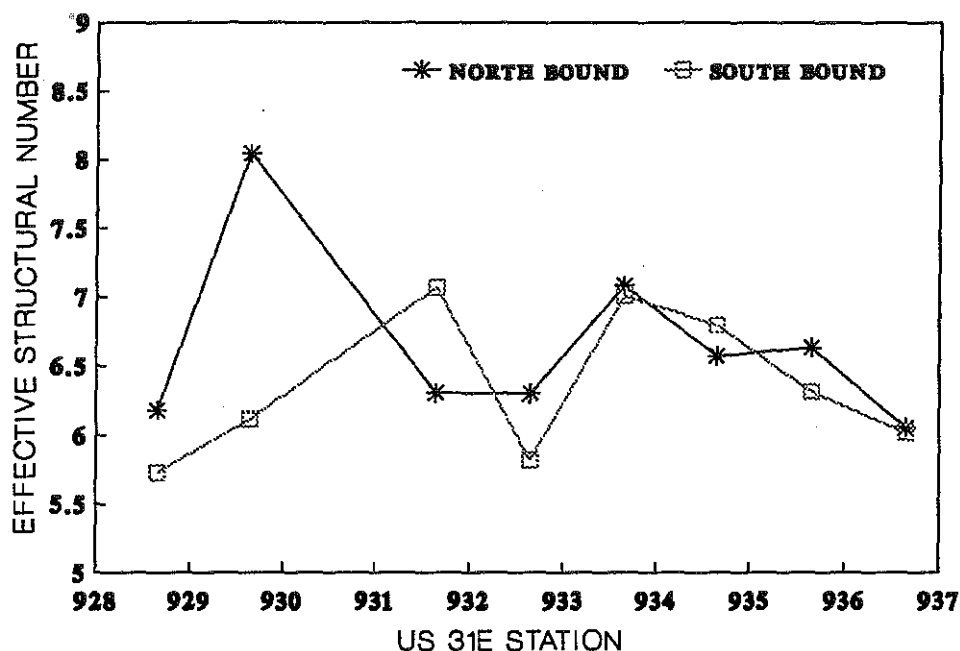


Figure 13. Pavement Structural Number calculated from FWD data.

Material properties reported by KYDOH personnel indicated on-site air contents ranging from 4.9 to 7.0 percent, slump ranging from 114 to 178 mm (4.5 to 7.0 in.), and an average 24-hour compressive strength of 2.50 MPa (3,630 psi). Results of material property tests reported by KYDOH are given in Table 5.

A lump sum contract was awarded to the contractor for this job. The lump sum bid for the project was \$72.95/m³ (\$61.00/yd³) for 1,204 m³ (1,440 yd³) or \$85,840.00.

Performance

A visual inspection of the PCCP inlay in June, 1995 indicated no distress. The concrete inlay was in excellent condition with no observed cracking of the PCCP slabs nor distress of the pavement joints.

Table 5. Data summary of PCC placed at intersection of US 62 and I-75 ramps.

<u>CYLINDER COMPRESSIVE STRENGTH</u>		
1 DAY	2.50 MPa (3,630 psi)	
25 DAY	4.39 MPa (6,370 psi)	
28 DAY	4.69 MPa (6,800 psi)	
<u>MEASURED AIR CONTENTS</u>		
10-21-94	6.7%	7.0%
10-22-94	4.9%	6.5%
<u>SLUMP TEST RESULTS</u>		
10-21-94	16.5 cm (6.5 in)	
	17.8 cm (7.0 in)	
10-22-94	11.4 cm (4.5 in)	
	14.0 cm (5.5 in)	

US 68 & Paris Bypass -- Bourbon County

The left turn lane from northbound US 68 to the Paris Bypass (south Paris) was removed and replaced with 305 mm (12 in.) of PCC pavement in 1981. The PCC pavement was not continuously reinforced but did include steel load transfer baskets at the joints. Prior to replacement, the turn lane had been a continuing problem requiring frequent rehabilitation for rutting and shoving. Unfortunately, details concerning pavement condition prior to PCC replacement and PCC design or material properties are not available for this site. Nor were there any information available regarding maintenance expenditures at the site or costs of the rehabilitation technique.

Performance

Since 1981, the adjacent bituminous pavement has been overlaid twice while no maintenance has been reported for the PCC turn lane. A visual inspection performed in June 1995 indicated little distress with minimal wear in the wheel paths, one mid-slab crack, and slight deterioration at the expansion joints. The 14-year performance of this section has been excellent.

US460/KY 114 & US 460 -- Magoffin County

The intersection of US 460/KY 114 and US 460 was rehabilitated in September, 1991. While the rest of the intersection was milled and resurfaced, the left turn lane from US 460/KY 114 eastbound onto US 460 eastbound was milled to a depth of 102 mm (4.0 in.). A high density plastic reinforcement grid (TENSAR® BX 140060) was placed on the base and a Class K base and a surface layer were placed above the grid. Approximately 14,632 m³ (17,500 yd³) of reinforcement was placed and the asphalt used in the bituminous concrete was AC 40. KYDOT personnel report no problems during construction.

Performance

The reinforced turn lane has performed well with minimal rutting and no reported rehabilitation since September 1991.

US 460 & US 23 -- Johnson County

In 1991, the intersection of US 460 and US 23 near Paintsville in Johnson County was overlaid. Reports from KYDOH indicate that the left turn lane from US 460 to US 23 northbound was milled to a depth of approximately 102 mm (4.0 in.). A TENSAR®

reinforcing grid was stapled to the base and bituminous base and surface courses were placed over the reinforcing grid. Reports indicate that by 1993 severe rutting had occurred to the point that traffic loops did not function properly. Also, the grid was apparently near the pavement surface in some areas. In 1993, the turn lane was milled to approximately the same depth as milled previously and a bituminous concrete with polymer additive was placed.

Performance

Interviews with KYDOH maintenance personnel indicate that as of June, 1995 no significant rutting has occurred in the turn lane. In comparing the relative effectiveness of grid reinforcement to polymer modified asphaltic concrete it should be noted that more loaded truck traffic existed when the grid was in place due to a detour established on US23 in Paintsville. However, the amount of additional traffic is not known.

Summary

Six intersections where experimental materials or techniques were utilized to address premature pavement failure are documented. Prior to reconstruction, all six of the intersections were constructed of conventional bituminous concrete pavements. Typical pavement failure mechanisms at intersections with bituminous pavement are rutting (vertical displacement) and shoving (lateral displacement). The mainline pavement type approaching all six of the intersections is bituminous concrete. At three of the intersections, a PCC inlay or PCC full depth replacement of the bituminous concrete was used and high density plastic reinforcement grids or polymer additives were placed in the bituminous concrete pavement structure or mixture at the remaining three sites evaluated during this study. Experimental activity at four of the intersections involved only specific lanes where pavement distress had required more frequent remedial action. Two intersections were designed for complete removal and reconstruction.

The evaluation period has not been sufficiently long at most of the sites to determine the effectiveness of the materials or techniques used, however, the PCC pavement has performed well in the three intersections where it was used. At Glasgow and Georgetown, the rigid pavement has been in service five years and one year respectively, with minimal distress. At Glasgow, there are a few mid-slab transverse cracks but most of the distress involves the pavement joints. Some deterioration of the pavement joint is occurring and the pavement seals may need replacing. The PCC turn lane at Paris in Bourbon County has been in service 14 years with little distress currently and no slab or joint maintenance

performed during this period. According to KYDOH personnel familiar with the project, the adjacent bituminous pavement has been overlaid twice during the 14 year period.

Experimental bituminous pavement intersections have met with varied success. The left turn lane at Salyersville in Magoffin County was reinforced with TENSAR® BX 140060. After nearly four years of service, distress is minimal and no particular construction problems were reported during the installation. The left turn lane at Paintsville in Johnson County was reinforced with a TENSAR® grid. However, after less than two years service, severe rutting necessitated rehabilitation of the turn lane. The lane was milled and a polymer additive was used in the bituminous concrete replacement. After two years, the turn lane is in good condition with only minimal rutting or shoving. One half of the bituminous concrete pavement at Glasgow in Barren County was reinforced with a TENSAR® grid. During the first 18 months of service, rutting was greater in the lanes with greater traffic volume and reinforcement appeared to have little effect. From 1994 to 1995, the rate of rutting in the reinforced lanes has decreased significantly while rutting in the non-reinforced lanes has continued. Rutting in several lanes is approaching 19 mm (0.75 in.) and the difference in the rate of rutting indicates the reinforcement grid is influencing that rate.

SUMMARY

The research reported herein focused on innovative attempts to alleviate prematurely distressed pavements at intersections. Bituminous pavements at intersections often exhibit excessive rutting, shoving and pushing, and washboarding early in their design lives. These distresses are attributed to high repetitive shear stresses on the pavement at locations where vehicles stop and start. As a result of the increased shear, these pavement sections require maintenance and/or rehabilitation early on in the life of the pavement. This early and continuing maintenance is costly. Significant savings may be realized if intersections and approaches are designed and constructed to accommodate the shear stresses as well as fatigue to which they are subjected. Pavements being rehabilitated should be designed to accommodate these higher stresses as well. Likely the greatest benefit of proper design considerations being given to intersection and approach pavements is that smoother pavements greatly enhance the complete safety of the transportation system, reduces the general discomfort to drivers and passengers, and alleviates bad public opinion for the traveling public.

The literature search focused on pavement distresses at intersections and methods used to upgrade intersection pavements. Methods most commonly used to upgrade an intersection

pavement to accommodate higher shear forces included: concrete overlays of asphaltic concrete pavements, or whitetopping; geogrid reinforcement of asphaltic concrete pavements; polymer-modified asphalts; stone mastic asphaltic concrete; and big-stone mixtures.

Whitetopping a bituminous concrete pavement has been proven to provide excellent performance. The excellent performance of a concrete overlay of an asphaltic concrete pavement was attributed to making use of the existing flexible pavement for underlying support. The overall safety of the pavement surface is increased by eliminating surface rutting.

Three intersection pavements were evaluated during this study wherein PCC was used in rehabilitation or reconstruction activities. One intersection involved a full-depth PCC slab in a turn lane where the bituminous pavement had required repeated maintenance. The 305-mm (12-in.) concrete turn lane is 14 years old and has not required any maintenance while the adjacent through lane has required at least two milling and overlay operations since the concrete turn lane was placed. Unfortunately, cost data for this project were not available. However, savings on maintenance activities and user costs are readily apparent due to the implementation of this rehabilitation technique. A second intersection where the bituminous pavement required repeated maintenance was replaced entirely with PCC pavement. Pavement design of the US 31E and US 68/KY80 intersection was 229 mm (9 in.) of PCC on 127 mm (5 in.) of untreated drainage blanket on 216 mm (8.5 in.) of DGA on a subgrade reinforced with high density plastic geogrid TENSAR® BX-100. Construction at the intersection proceeded in phases with only minimal closures or disruptions to traffic. Visual surveys of the intersection have been conducted annually. The latest visual survey of the 5-year old pavement indicated seven mid-slab transverse cracks. Several pavement joints exhibited missing sections of joint sealant and minor spalling and cracking. The joint distresses appear to be construction related and not load related however. The third PCC project involved milling the existing 102-mm (4-in.) bituminous pavement of US 62 at its intersection with I-75 in Scott County. A 102-mm (4-in.) PCC inlay was placed full width across four lanes. There were no problems reported during construction and performance of the nearly year old whitetop pavement has been excellent thus far. However, the evaluation period has not been sufficiently long at this site to determine the overall effectiveness of the materials or techniques used.

Experimental bituminous pavement intersections evaluated during this study have met with varied success. One half of the bituminous concrete pavement at Glasgow in Barren County was reinforced with a high-density plastic grid. The remaining portion of the high-density plastic geogrid was deleted from the project because of problems related to its installation. During the first 18 months of service, rut depths were greater in the lanes with

greater traffic volume and the geogrid reinforcement appeared to do little to impede the effects of traffic action on the pavement. However, from 1994 to 1995, the rate of rutting in the reinforced lanes has decreased significantly while rutting in the non-reinforced lanes has continued. Rutting in several of the non-reinforced lanes is approaching 19 mm (0.75 in.) and the difference in the rate of rutting indicates the reinforcement grid is influencing that rate. The left turn lane at Salyersville in Magoffin County was reinforced with TENSAR® BX 140060. KYDOH personnel reported no particular problems with the installation of the plastic geogrid. Observed distresses have been minimal after nearly four years of service. The left turn lane of US 460 at Paintsville in Johnson County was also reinforced with a TENSAR® grid. However, after less than two years service, severe rutting necessitated rehabilitation of the turn lane. It should be noted that during this time, significantly more heavy-truck traffic used this route because of a detour on US 23. Because of the failure, the turn lane was milled and a polymer additive was used in the bituminous concrete that was replaced. After two years, the turn lane has performed well and is in good condition with only minimal rutting or shoving.

There have been numerous projects incorporating polymer-modified asphalts in Kentucky and those projects have met with varying success. Evaluations of the polymer-modified asphalts were conducted to determine whether the asphalt additives would enhance performance of the materials and prolong pavement life. Evaluations generally included construction inspections, performance inspections, and laboratory and field investigations of the modified asphaltic concrete materials. Results of the assorted investigations have varied. Laboratory investigations have demonstrated that polymer-modified mixtures have less potential for rutting compared to conventional mixes. Experimental uses of polymer-modified asphaltic concrete were recommended for performance monitoring under the long-term monitoring study. Sufficient long-term performance data of the polymer-modified asphalt projects have not been obtained to provide verification of the laboratory tests. Nevertheless, the use of polymer-modified asphaltic concrete mixtures has generally proliferated throughout the state and with little assessment of its performance.

Task D of the work plan called for economic analyses of the various strategies used at the intersections to minimize pavement distresses. The major thrust of this effort was in obtaining cost data for construction and maintenance activities performed at the intersections evaluated. Researchers were unable to specifically determine certain costs related to past maintenance activities at the intersections evaluated. Unfortunately, these costs were critical to the completion of Task D. Long-term performance data are necessary as well to determine benefit-cost ratios of the rehabilitation techniques. The long-term performance of any rehabilitation technique must be adequately sufficient to offset the expected increase in the initial cost associated with using more sophisticated materials and procedures. Ideally, the annual benefits to the users would be compared to the annual costs

to the state to properly maintain the intersection pavement. To compare continuing maintenance to a particular rehabilitation strategy, cost data from past maintenance activities would be obtained, a reasonable rate of return assumed, and an annual cost determined for some economic life. An annual cost of the maintenance activities performed at the intersection over a definite time period would be compared to an annual cost (first cost spread out over the same time period) of the particular rehabilitation technique employed. The annual-cost method of comparison is one of the most widely employed methods of evaluation and is based on the conversion of the cost of each alternative into an equivalent uniform series of payments.

An illustration of annual cost for a rehabilitation strategy that would eliminate significant maintenance expenditures on intersection approaches would be beneficial to any conclusions reached regarding a particular rehabilitation strategy. However, information relative to performance and expected annual maintenance expenditures for the treatment types are needed to perform the analysis. Further, an overlay interval for the bituminous pavement would have to be assumed, if not known. The overlay interval will depend on traffic volumes and loadings and on the bituminous pavement rehabilitation treatment (whether a modified asphalt or geogrid was used). When maintenance is required mobilization, demobilization, and maintaining and controlling traffic contribute greatly to the total cost of the rehabilitation.

CONCLUSIONS AND RECOMMENDATIONS

Based on the excellent performance of the concrete sections observed during this study, it is recommended that the use of PCCP be encouraged whenever maintenance expenditures at intersections are excessive or where new intersections are being constructed that would experience significant traffic volumes and loadings. The performance of PCC overlays on an asphaltic concrete pavement have demonstrated their viability. There are the only minimal surface preparations involved and the PCC overlay utilizes the existing flexible pavement for underlying base support. Portland cement concrete overlays minimize maintenance expenditures and increase the overall safety at the intersection by eliminating surface distortions.

The use of high-strength, polypropylene grids can be regarded as one way to reinforce an asphaltic concrete pavement to diminish the distressing effects of traffic such as rutting and shoving. The key to using these high-strength materials for reinforcement efficiently lies in the ability of the contractor to suitably install the grid such that it can provide added tension to the asphaltic concrete pavement. Based on the investigations conducted during this

study, high-density plastic geogrids appear to provide some resistance to long-term permanent deformation of the asphaltic concrete pavement. It is recommended that high-density plastic reinforcing grids be considered as a rehabilitation option for distressed intersection pavements. The Engineer must provide special attention to the installation of the plastic grid. The correct installation procedure must be specified in the contract document and the Engineer must work closely with the contractor to ensure the grid is correctly installed without difficulties (Appendix C of this report contains the Special Note for Construction for bituminous pavement reinforcement).

KYDOH has utilized an extensive number of asphaltic concrete modifiers. However, there has been little, if any, follow-up with regard to performance monitoring. These experimental sections must be properly evaluated for long-term performance to document the effectiveness or ineffectiveness of the modifiers. It is recommended that KYDOH initiate an investigation to determine where modifiers have been used, the types of modifiers that have been used, any costs associated with their use, and the effectiveness or ineffectiveness of the application. Warrants necessary to implement the use of the most effective asphalt modifiers may be developed from information gained through this investigation.

Stone matrix asphalt pavements have reportedly provided rut-resistant pavements that have extended pavement life as much as 25 to 30 percent. Performance data for SMA sites in the United States are not yet available. It is recommended that KYDOH follow the lead of the states that have implemented this European technology. If the SMA experiments in surrounding states receive favorable reviews, then Kentucky should attempt a field trial of its own to determine the merits of this system.

Large stone mixtures are being used increasingly in Kentucky. These mixtures appear to have increased shear resistance and provide resistance to increased tire pressures and wheel loads. It is recommended that KYDOH evaluate the performance of Class K base at an intersection to determine the validity of this application. This could be achieved either through rehabilitation or new construction.

Intersection pavements should no longer be designed and constructed as main line sections. It is quite possible that intersection pavements and their approaches might someday be considered as a separate bid item in a construction project and include different materials specifications and be designed for maximum shear conditions.

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Appendix A

**SPECIAL NOTE FOR
PORTLAND CEMENT CONCRETE PAVEMENT/24/48/72
(EXPERIMENTAL)**

**SPECIAL NOTE FOR
PORTLAND CEMENT CONCRETE PAVEMENT/24/48/72
(EXPERIMENTAL)**

I. DESCRIPTION

This specification covers portland cement concrete pavement constructed using mixtures capable of attaining 3,500 pounds per square inch compressive strength and being opened to traffic within: 24 hours for PCCP/24; 48 hours for PCCP/48; 72 hours for PCCP/72; after placing and finishing. The type of mixture will be specified elsewhere in the contract.

All requirements in the Standard Specifications, plans, or proposal related to portland cement concrete pavement shall apply except as specifically superseded herein. Section references herein are to the Department's 1988 Standard Specifications.

II. MATERIALS

All materials shall conform to the requirements specified in Section 501.02. Type III cement will be permitted.

When application of water to the surface of the pavement is permitted, it shall be applied as a fog spray by means of approved spray equipment.

III. MIX DESIGNS

A. General. The concrete mixture shall attain 3,500 psi compressive strength within the specified time. The design in paragraph III.B herein should be capable of producing the required strength. Other designs may be proposed by the Contractor but shall be reviewed and approved by the Engineer. However, the Contractor is responsible for furnishing concrete meeting the specified strength with either mix design. Regardless of which design is used, the contractor shall be responsible for making trial batches as necessary to demonstrate that the mixture will meet the requirements for slump, air content, water/cement ratio, and compressive strength. The trial batches shall be made using the ingredients, proportions and equipment (including batching, mixing and delivery time) to be used on the project. At least two consecutive trial batches meeting all specified requirements shall be made. Department personnel shall observe all phases of the trial batches. A report containing mix proportions and test results for slump, air content, water/cement ratio, and compressive strength shall be submitted for each trial batch for review and approval by the Engineer.

B. Design. The mixture may be one of the following as applicable:

<u>MIX</u>	<u>MINIMUM</u>	<u>MAXIMUM</u>	<u>ADMIXTURE DOSAGE RANGE</u>
	<u>CEMENT FACTOR</u>	<u>FREE WATER</u>	<u>TYPE F OR G</u>
	<u>BAGS/YD³</u>	<u>GALS/BAG(LB/LB)</u>	<u>OZ/100 LBS. CEMENT</u>
PCCP/24	8.50	3.75 (.333)	12-25
PCCP/48	7.75	3.88 (.344)	12-25
PCCP/72	7.00	4.00 (.355)	12-25

(1) The slump range may be 2 to 5 inches except when forms are used the slump may be up to 7 inches. Edge slump requirements of Section 501.24.01 of the Standard Specifications still apply.

(2) The mixture shall have a net entrained air content, by volume, of 6.5% plus or minus 1.5%.

C. Other Mixtures. If the Contractor proposes an alternate design, the following requirements shall be met.

(1) The cement content shall not be less than seven bags per cubic yard.

(2) The mixture shall have a net entrained air content, by volume, of 6.5% plus or minus 1.5%.

IV. STRENGTH TESTING AND OPENING TO TRAFFIC

A. Compressive Strength Specimens. Four (two sets) of 6-inch by 12-inch cylinders will be cast from each 150 cubic yards of concrete. Casting, curing, and testing of cylinders will be performed by Department personnel in accordance with standard procedures.

B. Compressive Strength Testing for Opening to Traffic. One set of cylinders will be tested from each 150 cubic yards no later than: 24 hours for PCCP/24; 48 hours for PCCP/48; 72 hours for PCCP/72; plus or minus 1 hour from time of molding. The resulting average compressive strength will dictate one of the following actions:

(1) If the average compressive strength is 3,500 psi or above, the pavement may be opened to traffic and the remaining set of cylinders discarded.

(2) If the average compressive strength is less than 3,500 psi, the remaining set of cylinders will be tested, at a time or at an age that is appropriate in the judgment of the Engineer, based upon the closeness of the first set to the required strength.

(3) If the second set of cylinders tests less than 3,500 psi compressive strength, the concrete pavement shall not be opened to traffic until it has attained an age of seven days, unless otherwise directed. In this event, acceptance of the pavement will be determined based on additional testing as directed by the Engineer.

C. Consecutive Test Requirements. When two consecutive first sets of cylinders or when two first sets out of any four first sets of cylinders do not reach 3,500 psi compressive strength, the work will be suspended until a satisfactory mix design adjustment is proposed by the Contractor. The adjustment shall be made in subsequent production mixture; additional trial batches are not required.

D. Opening to Traffic. All required work, including but not limited to joint sawing, joint sealing, shoulder or curb construction, and sweeping and cleaning the pavement shall be coordinated so the pavement is ready to open to traffic when the specified strength is attained.

V. MEASUREMENT AND PAYMENT

Measurement and payment will be in accordance with Sections 501.27 and 501.28, which shall be full compensation for all work required to construct the pavement.

Payment will be made under:

<i>Pay Item</i>	<i>Pay Unit</i>
(Thickness) PCC Pavement/24	Square Yard
(Thickness) PCC Pavement/48	Square Yard
(Thickness) PCC Pavement/72	Square Yard

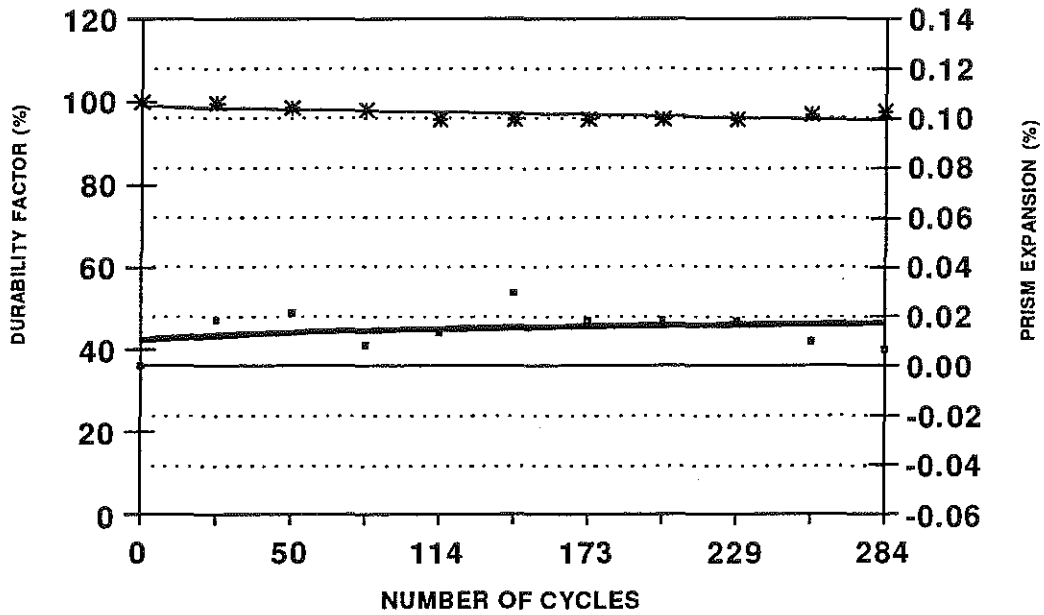
June 15, 1990

APPENDIX B

**RESULTS OF
FREEZE / THAW TESTING**

GLASGOW BYPASS

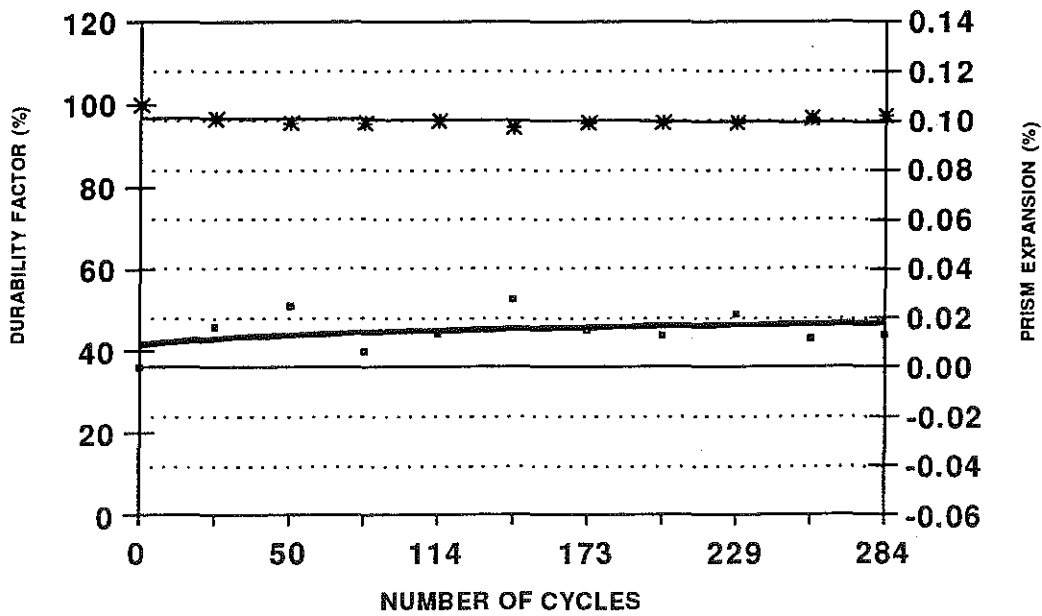
**GLASGOW BYPASS SITE
RESULTS OF FREEZE/THAW TESTS**



* DURABILITY — EXPANSION

SPECIMEN SERIES G

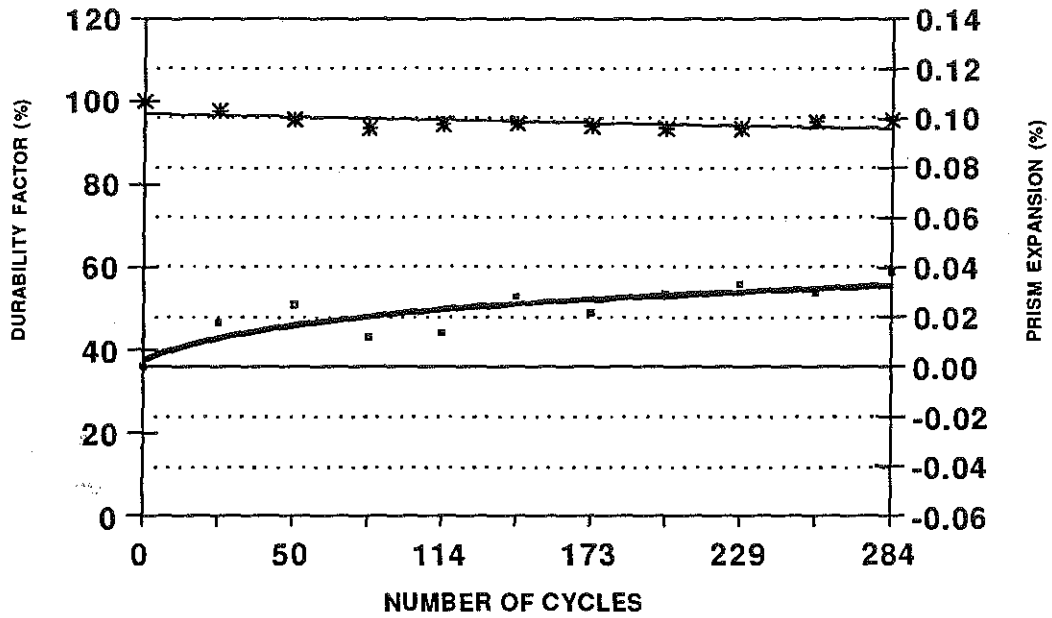
**GLASGOW BYPASS SITE
RESULTS OF FREEZE/THAW TESTS**



* DURABILITY — EXPANSION

SPECIMEN SERIES G-2

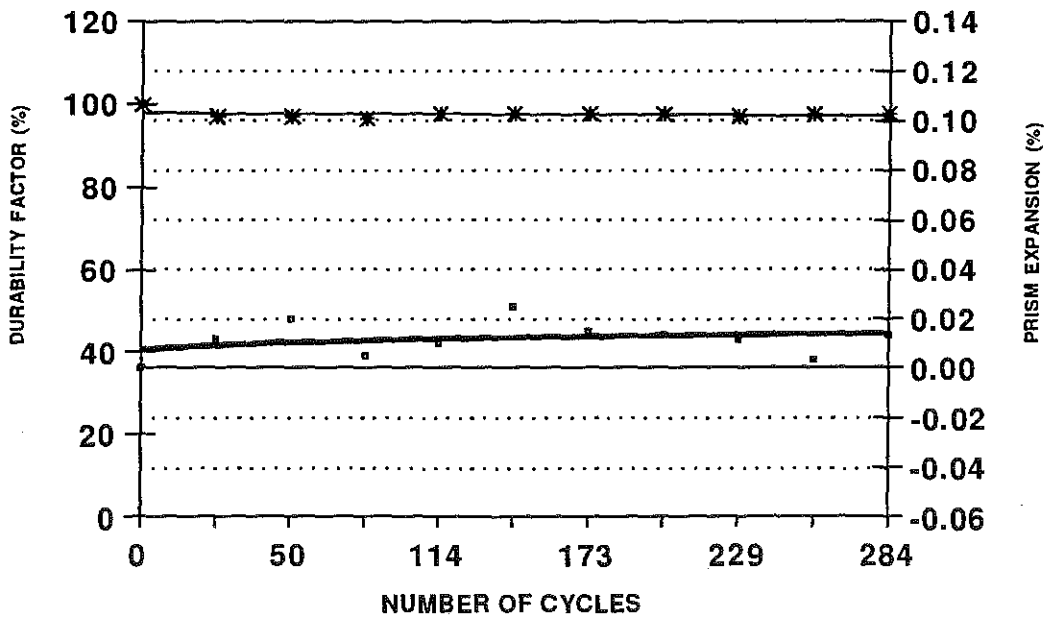
**GLASGOW BYPASS SITE
RESULTS OF FREEZE/THAW TESTS**



* DURABILITY — EXPANSION

SPECIMEN SERIES G-3

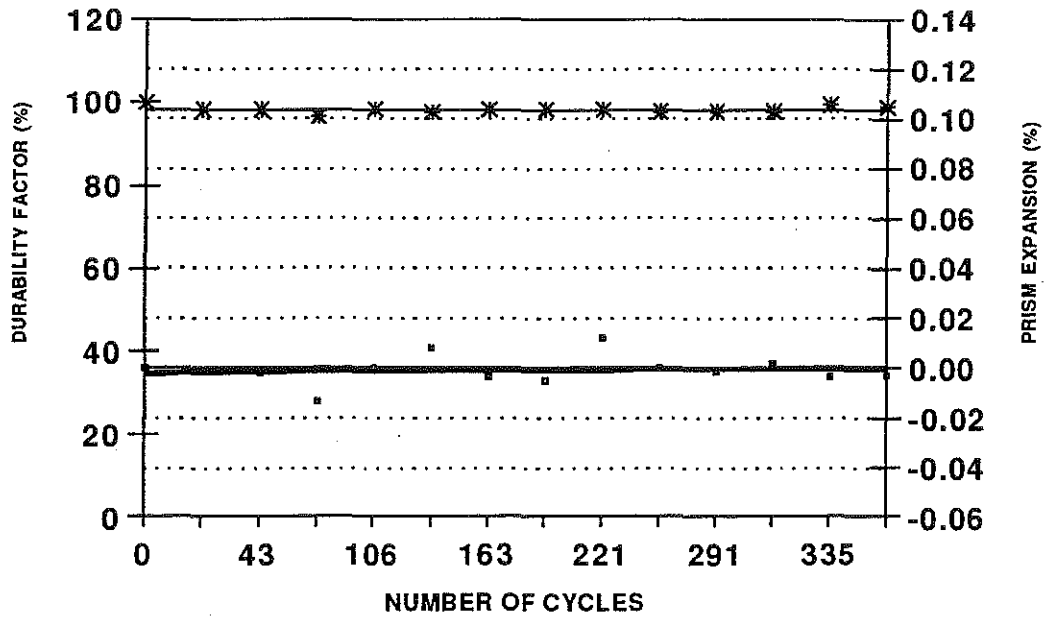
**GLASGOW BYPASS SITE
RESULTS OF FREEZE/THAW TESTS**



* DURABILITY — EXPANSION

SPECIMEN SERIES G-4

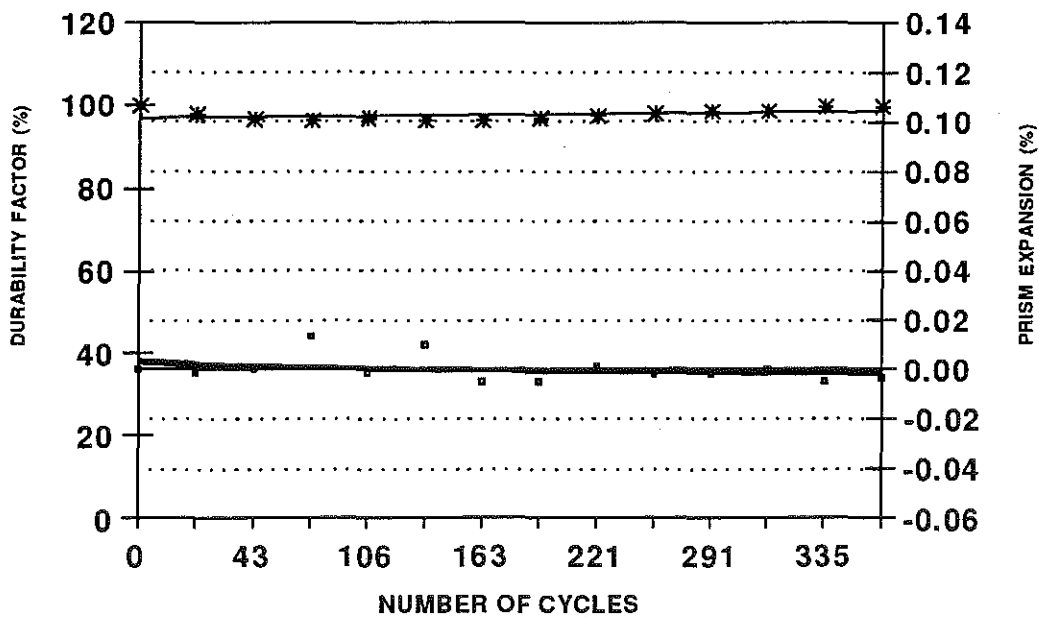
**GLASGOW BYPASS SITE
RESULTS OF FREEZE/THAW TESTS**



* DURABILITY — EXPANSION

SPECIMEN SERIES G'-A

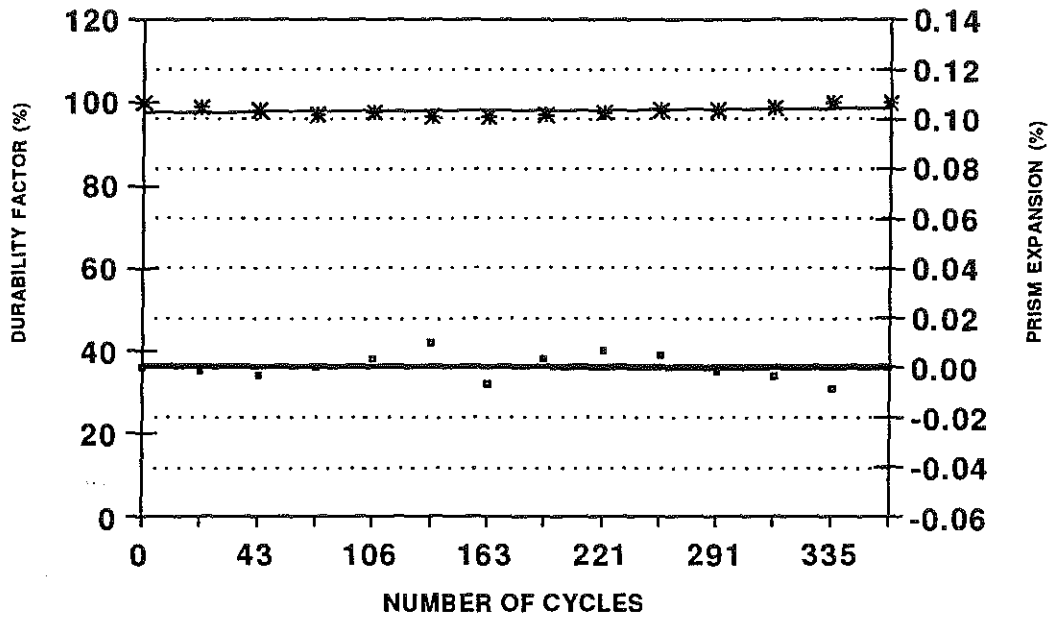
**GLASGOW BYPASS SITE
RESULTS OF FREEZE/THAW TESTS**



* DURABILITY — EXPANSION

SPECIMEN SERIES G'-B

**GLASGOW BYPASS SITE
RESULTS OF FREEZE/THAW TESTS**



* DURABILITY — EXPANSION

SPECIMEN SERIES G'-C

Appendix C

**SPECIAL NOTE FOR
BITUMINOUS PAVEMENT REINFORCEMENT
(EXPERIMENTAL)**

**SPECIAL NOTE FOR
BITUMINOUS PAVEMENT REINFORCEMENT
(EXPERIMENTAL)**

I. DESCRIPTION

This note covers requirements for installation of a geogrid between layers of bituminous concrete base, as reinforcement of the base.

II. MATERIAL

The reinforcement shall be the latest design of TENSAR® Corporation's Biaxial Geogrid BX 1400. The Engineer may obtain samples of material delivered to the project for informational testing.

III. CONSTRUCTION REQUIREMENTS

A. General. Before beginning installation, the Contractor shall furnish to the Engineer on the project copies of the manufacturer's literature and specifications showing details of the reinforcement material and methods of installation. The manufacturer's literature shall include complete instructions for cutting, placing, lapping, tensioning, and installing the material.

A representative of the manufacturer shall be on the project when work begins and shall remain on call as the project progresses to advise the Engineer. Any costs of the manufacturer's representative visiting the project will be considered incidental to construction of the Bituminous Pavement Reinforcement.

B. Reinforcement Installation. The reinforcement shall be installed on the bituminous concrete base course as shown on the typical section. The entire area of the base course shall be covered within the limits designated. Sweeping of the base will be required if dirt or debris is present.

The specified tack coat shall be applied before the geogrid is installed.

The grid shall be cut, placed, overlapped, tensioned, and fixed in place in strict accordance with the manufacturer's written instructions.

Except for paving equipment and vehicles, no traffic will be allowed on the reinforcement until the following course of bituminous concrete base is placed.

C. Paving. The bituminous concrete base placed over the grid shall be placed and compacted in a conventional manner. Care shall be taken in vehicle movements and paver operation to ensure that the grid is not disturbed.

Compaction requirements specified elsewhere in the contract shall apply. Caution shall be exercised in operation of rollers to prevent movement of the grid.

D. Fault Repair. Any visible faults that occur due to movement of the grid shall be repaired immediately after rolling. For small areas, paving mixture shall be removed from the area of the fault, the grid replaced in its original position, and the base mixture replaced, leveled, and compacted. The grid may be cut if necessary for it to lie flat.

Large areas of unsatisfactory work shall be removed and replaced.

IV. METHOD OF MEASUREMENT

Bituminous Pavement Reinforcement will be measured in square yards, computed as the total area of pavement acceptably reinforced as measured in place.

Other pavement items will be measured and paid as specified elsewhere in the contract.

V. BASIS OF PAYMENT

Payment for the accepted quantity of Bituminous Pavement Reinforcement will be made at the contract unit price. Such payment shall be full compensation for all labor, materials, equipment, and incidentals necessary to complete the work.

Payment will be made under:

Pay Item	Pay Unit
Bituminous Pavement Reinforcement	Square Yard

December 19, 1989

Appendix D

General Notes

Georgetown-Cynthiana Road (US 62)

MAINTENANCE OF TRAFFIC

SCOTT COUNTY
GEORGETOWN-CYNTHIANA ROAD (US 62)
07105 0062 009-011

GENERAL NOTES

TRAFFIC CONTROL

1. Traffic shall be maintained in accordance with the current editions of the *Manual on Uniform Traffic Control Devices*, the *Standard Specifications for Road and Bridge Construction* and the *Standard Drawings*.
2. Except for the roadway and traffic control bid items listed, all items of work necessary to maintain and control traffic will be paid for at the lump bid price to "Maintain and Control Traffic" as set forth in the current *Standard Specifications for Road and Bridge Construction* unless otherwise provided for in these notes. The lump sum bid to "Maintain and Control Traffic" shall also include, but is not limited to, the following items and operations:
 - a. All grading and necessary drainage (*unless a bid item for detour construction is included*) for the temporary roadway and removal thereof when it is no longer needed. If a bid item for detour construction is included, grading and drainage will be paid for in the bid item "Detour Construction".
 - b. All labor and materials necessary for construction and maintenance of traffic control devices and markings.
 - c. All flagpersons and traffic control devices such as, but not limited to, flashers, signs, barricades and vertical panels, plastic drums (*steel drums will not be permitted*) and cones necessary for the control and protection of vehicular and pedestrian traffic as specified in these notes, the plans, the *Manual on Uniform Traffic Control Devices* or the Engineer.
3. Any temporary traffic control items, devices, materials and incidentals shall remain the property of the Contractor, unless otherwise addressed, when no longer needed.
4. The contractor shall maintain a two-lane traveled way with a minimum lane width of 10 feet. However, during working hours one-way traffic may be allowed at the discretion of the Engineer provided adequate signing and flagpersons are at the location.
5. The Contractor shall completely cover any signs, either existing, permanent, or temporary which do not properly apply to the current traffic phasing, and shall maintain the covering until the signs are applicable or are removed.

6. In general, all traffic control devices shall be placed starting and proceeding in the direction of the flow of traffic and removed starting and proceeding in the direction opposite to the flow of traffic.
7. The Engineer and the Contractor, or their authorized representatives, shall review the signing before traffic is allowed to use any lane closures, crossovers or detours. All signing shall be approved by the Engineer before work can be started by the Contractor.
8. If the Contractor desires to deviate from the traffic control scheme and construction schedule outlined in these plans and this proposal, he shall prepare an alternate plan and present it in writing to the Engineer. This alternate plan can be used only after review and approval of the Divisions of Traffic, Design and Construction and the Federal Highway Administration, where applicable.
9. If traffic should be stopped due to construction operations and an emergency vehicle on an official emergency run arrives on the scene, the Contractor shall make provisions for the passage of that vehicle as quickly as possible.
10. No lane shall be closed for more than one week.
11. The Department of Highways shall perform all striping.

PROJECT PHASING

Phase I:

The two turning lanes PCCP/24 will be constructed.

Phase II:

The left lane PCCP/24 will be constructed and the left turn lane will be open. Use of a "Left Turn Only" sign (with an arrow) will be required.

Phase III:

The right lane PCCP/24 will be constructed.

CONTRACTOR'S VEHICLES

The Contractor's vehicles shall always move with and not against the flow of traffic. Vehicles shall enter and leave work areas in a manner which will not be hazardous to or interfere with normal traffic. Vehicles shall not park or stop except within work areas designated by the Engineer.

SPECIAL NOTES
SCOTT COUNTY
07105-0062-009-011

1. The 4-inch PCC Pavement Inlay shall be portland cement concrete pavement constructed using mixtures capable of attaining 3500 psi compressive strength and being opened to traffic within 24 hours (PCCP/24). Maximum size aggregate shall be 1 inch.
2. Bituminous pavement milling and texturing to a depth allowing for the 4-inch PCC Pavement Inlay shall be incidental to the unit bid price for the 4-inch PCCP/24 Inlay and as directed by the project engineer.
3. The removal and installation of traffic loops as directed by the project engineer shall be incidental to the unit bid price for the 4-inch PCCP/24 Inlay. The traffic loops shall be installed prior to placement of the 4-inch PCCP/24 Inlay.
4. The removal of the TYPE V pavement markers shall be incidental to the unit bid price for the 4-inch PCCP/24 Inlay.
5. Maintain and Control Traffic shall be incidental to the unit bid price for the 4-inch PCCP/24 Inlay.
6. Either central-mixing or truck-mixing will be permitted (at the contractor's option) in cement concrete pavement construction. Hand finishing will be permitted.
7. All transverse contraction and expansion joints shall be at right angles and shall be spaced at 6 feet intervals as directed by the project engineer.
8. Longitudinal joints shall be spaced at 6 feet intervals.
9. Saw joints conventionally or with soft cut saw to a depth of 1 inch. Joints shall be sealed with Dow Corning silicone 890 SL or equivalent.
10. Traffic shall not be permitted on newly sealed joints until the silicone seal is sufficiently "skinned over" to prevent tracking due to traffic. The "skin over time" for silicone seals typically is one hour, however, longer times may be required depending upon specific weather conditions. Joints shall be sealed in accordance with current standard specifications and standard drawings. The contractor shall be responsible for replacement/repair of damaged seals until final curing is complete (21 days).
11. Before placing the PCCP/24, the bituminous pavement shall be saw cut to provide a neat clean edge.
12. The method of placement for the PCCP/24 shall be approved by the Engineer in Construction prior to beginning paving operations. Recommended slump for PCCP/24 is 3 inches.

13. Contrary to the Special Note For Portland Cement Concrete Pavement/24/48/72 (Experimental), whenever the ambient air temperature is below 60 degrees F, the PCCP/24 shall be covered with insulating blankets. No other type of insulation will be permitted.
14. Synthetic fibers shall be added to the PCCP 24/48/72 mixture at the plant as recommended by the manufacturer. The fibers shall be graded, fibrillated, polypropylene fibers and shall be added to the fresh concrete mixture at a dosage rate of 3 pounds per cubic yard of concrete. The cost of the fibers and any additional labor cost shall be included in the bid price for the PCCP24 Concrete.

