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AN EVALUATION OF THE COST EFFECTIVENESS OF D-CRACKING PREVENTIVE MEASURES
ODOT Agreement No. 8772

SUMMARY REPORT

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For

Ohio Department of Transportation

September 2001

Executive Summary

Title: An Evaluation of the Cost Effectiveness of D-Cracking Preventive Measures

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D-cracking has long been a serious problem in the deterioration of concrete pavements in severe weather climates. After much research, the mechanics and variables involved in the destructive forces of concrete D-cracking are becoming known. This study focused on these variables and included analysis of the cost effectiveness of using certain preventive measures to reduce premature deterioration of concrete pavement due to D-cracking.

A test road located on State Road (SR) 2 near Vermilion, Ohio was built in 1974 and 1975 with specific sections to investigate the role of subbase drainage systems, pavement joint design, subbase materials, joint sealant, different aggregate sources and size, different cements, types of cure, and joint spacing. In 1998 this field study was done on the Vermilion project to evaluate many of the factors that were initiated on the pavement. The significant findings are as follows:

- Aggregate size was not a determining factor of total quantity of bad pavement. With small aggregate the problem was mid-slab cracking. With large aggregate the problem is joint deterioration due to D-cracking. The net effect was either having deteriorated pavement at the joints (large aggregate) or at the mid-slab (small aggregate).
- Aggregate source was a determining factor of the total quantity of bad pavement.
- To reduce D-cracking, a high quality and/or small sized coarse aggregate should be used.
- To reduce mid-slab cracking, larger aggregate sizes should be used.
- Use of vapor barriers does not increase pavement performance.
- Subbases with drains performed about the same as ones without drainage.
- The use of no joint sealant prevented joint deterioration due to D-cracking as well or better than hot pour or neoprene. Neoprene was significantly worse.

The Vermilion test pavement did provide a verification of useful information regarding the contributions different variables make towards maintaining the integrity of both joints and pavement.

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16. Abstract

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Prepared in cooperation with the Ohio Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

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ABSTRACT

D-cracking has long been a serious problem in the deterioration of concrete pavements in severe weather climates. After much research, the mechanics and variables involved in the destructive forces of concrete D-cracking are becoming known. This study focuses on these variables that include analysis of the cost effectiveness in using certain preventive measures to reduce premature deterioration of concrete pavement due to D-cracking. These variables will include aggregate source, cement source, joints, type of pavement, vapor barrier, cure, and subbase.

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CHAPTER I. INTRODUCTION

Context of the Problem

Laboratory studies have found that reducing the particle size of susceptible coarse aggregates will greatly improve the durability of concrete exposed to damaging freeze-thaw conditions. A test road located on State Road (SR) 2 near Vermilion, Ohio was built in 1974 and 1975 with specific sections to investigate the role of subbase drainage systems, pavement joint design, subbase materials, and joint sealant (variables that affect the availability of moisture to the pavement slab). Other possible variables that might affect the quality of different pavement sections of slab construction were introduced. These included different aggregate sources and size, different cements, type of cure, and joint spacing. This test road afforded an opportunity to better test findings and conclusions drawn from previous laboratory findings on D-cracking under actual service conditions. The road sections were visibly inspected annually, and development of D-cracking, popouts, transverse cracks, pumping, and faulting were all noted.

D-cracking is the number one cause of concrete deterioration in the Midwest and has been a very expensive problem in the State of Ohio. Coarse aggregate quality is the primary component of D-cracking. Therefore, an investigation of the role of coarse aggregate size and quality is warranted. This will include evaluating the role of various aggregates on D-cracking deterioration on the SR2 test pavement.

Objectives of the Research

(as stated in the original research proposal)

The objective of this paper will be to do:

1. A literature search on solutions to D-cracking in general.
2. A background evaluation of the Vermilion SR 2 project.
3. An assessment of the effectiveness of various D-cracking preventative measures.
4. A cost evaluation of the effectiveness of these measures.

The evaluation will compare the cost of implementing D-cracking preventive measures versus not implementing D-cracking preventive measures. The cost of not implementing D-cracking preventive measures includes no additional cost for the initial construction but is offset by future anticipated repairs. In this case, that information is available not only from the 1993 repairs of the Vermilion project, but also from similar repair projects. The other design option is to implement D-cracking preventive measures in anticipation of reducing future repairs. Specifically, the cost effectiveness of using smaller aggregate sizes will be investigated as D-cracking preventive measures and reduced joint spacing will be investigated as a general preventive measure for overall pavement deterioration.

This will involve comparing the cost of the inclusion of the preventive D-cracking measures into the initial construction, versus the cost of subsequent pavement repair resulting from these initial design decisions. Aggregate size and source will be the focus of the preventive measures. The subsequent pavement repair cost analysis will evaluate the incidence of D-cracking deterioration, the increase in number of transverse cracks due to the use of the smaller aggregates, and the possible problems created by shorter slab lengths.

The source of information for this analysis will be the documentation from ODOT Project 6000(92). The incidence of deterioration will be determined from a review of the project documents and job site visits in which the actual repairs will be plotted. The costs of these repairs will be determined from both the initial bid documents and any subsequent change orders due to actual job conditions. Questions to be answered:

1. What is the cost of requiring only large size, high quality aggregates be used?
2. Smaller size aggregates are known to prevent D-cracking, but does this solution create more problems than it solves?
3. What is the cost of the transverse cracking that results from the use of smaller aggregate?
4. What is the cost of constructing and maintaining shorter slabs in an effort to prevent D-cracking?
5. What is the market effect of requiring only large size aggregates?

Anticipated Benefits

The primary benefit of this study will be to measure whether the D-cracking preventive methods were cost effective and give recommendations on how such measures can be applied in the future. If the measures used to prevent D-cracking are found to be cost effective in the long term, then instituting these measures as a matter of policy could generate great savings for ODOT.

CHAPTER II. REVIEW OF LITERATURE

Definitions of "D-Cracking"

The term "D-cracking" dates back to the 1930s and has since come to mean different things to different authors and observers. To some, it refers to any crack in concrete filled with a secondary deposit (such as eggringite), regardless of the cause of the cracking (Paxton, 1974). Others use the term in reference to cracks that have resulted from the freezing and thawing ("weathering") of concrete, which may or may not have secondary deposits. In the research by ODOT, D-cracking refers to the series of closely spaced cracks that appear at a pavement surface adjacent to and roughly parallel to transverse and longitudinal joints and cracks, and the free edges of pavement slabs. It also refers to associated cracking preliminary to that appearing at the wearing surface (Paxton, 1974) (noted only when cores are taken from a pavement slab). D-cracking is defined by a characteristic "hour glass" crack pattern, which indicates a particular type of distress.

D-cracking normally first appears at transverse joint intersections, and occasionally at the intersection of longitudinal joints and transverse cracks at the outside corners of a pavement slab. As the cracking progresses through the joint, the resulting cracks form a distinctive pattern easily identified, as the cracks radiate from the joint and/or pavement edge.

Laboratory Observations

Laboratory observations of D-cracking are typically done by examining cores of pavement slabs. D-cracking appearing at the surface is a manifestation of deterioration that usually originates in the lower and middle levels of the pavement slab and progresses upward to the wearing surface. By the time the cracking appears at the surface, much damage has already been done at the subsurface. In the early stages of deterioration prior to its appearance at the surface, the pattern characteristically consists of a series of mostly horizontal cracks that are confined to the lower levels of the slab. Eventually, these cracks become enlarged and additional microcracks form, resulting in complete deterioration of the lower levels of the slab. A number of cracks in the upper levels of the slab may extend to the wearing surface, where the deterioration can finally be noted by observation. Eventually, the cracks extend and enlarge throughout the slab, destroying it.

Past Research on D-Cracking

Because D-cracking has been a perennial problem in the Midwestern United States, it has been the basis for a great deal of research. However, much of this literature has focused on the mechanics of D-cracking, oriented towards the engineered properties of concrete. Of the 23 citations available from Ohiolink, only 5 pertained to D-cracking and pavement performance. The remaining citations concerns peripheral subjects such as testing of aggregate, mechanics of water freezing within the coarse aggregate, the effects of chemical reactivity of concrete components on D-cracking, and other topics more closely related to the engineering aspects of concrete construction. This same pattern occurred when performing a computerized dissertation search with the keyword D-cracking. This search looked at all abstracts of dissertations written from 1861, and listed five dissertations with "D-cracking" appearing somewhere in the abstract. Of these, four dissertations concerned testing of aggregates to determine D-cracking susceptibility, and one dissertation concerned asphalt overlaying D-cracked pavement. Because these dissertations did not relate to D-cracking preventive measures, they will not be further discussed.

Next, an annotated bibliography was obtained from Dr. Mark Snyder of the University of Minnesota. This bibliography included citations, case histories, and laboratory investigations related to all aspects of D-cracking. Included also are peripheral topics relating to D-cracking, as well as contributing works from such fields as geology, physics, and soil physics. From this annotated bibliography, all references that were not focused chiefly on engineering perspectives of D-cracking were obtained from the Bowling Green State University Library.

Ohio Reports

The first documents examined were the reports related to the initial research of D-cracking in Ohio by Construction Technology Laboratories in conjunction with the Ohio Department of Transportation. These documents were: 1) D-cracking of Concrete Pavements in Ohio by Klieger, Monfore, Stark, and Teske (1974). 2) The Influence of Environment and Materials on D-Cracking by Klieger, Monfore, Stark, and Teske (1978). 3) The Significance of Pavement Design and Materials in D-Cracking by Stark (1986).

The first of these, D-cracking of Concrete Pavements in Ohio, was the first close examination of portland cement concrete highways in the state of Ohio. The report was the culmination of a four-year research program initiated in 1969 to determine the extent and severity of D-cracking, to identify factors affecting its rate of development, and find means of preventing its development in future construction. A field survey of concrete highways in the state found that a total of 18% of all joints suffered from D-cracking deterioration, presenting a serious problem to the state. Overall, D-cracking was more prevalent in the southwest part of the state, and least prevalent in the northwest part of the state due to geographic differences, while the greatest severity of deterioration was found in the Mount Vernon-Lancaster-Zanesville area. In its conclusion, this report introduces a proposed experimental pavement to be built into a four-mile section of State Route 2 in Vermilion that was the subject of this research. Overall, D-Cracking of Concrete Pavements in Ohio established the foundation of this project, with the exception of its general background of D-cracking history on Ohio pavements. However, this report does introduce and discuss the relationship between a reduction in aggregate size and a corresponding reduction in D-cracking.

The next report prepared was The Influence of Environment and Materials on D-Cracking, dated October 1978. This report on the research program initiated by the Ohio Department of Transportation and the Portland Cement Association had three stated objectives:

1. To monitor the performance of selected pavements with respect to the effect of moisture content and reduced aggregate particle sizes on the development of D-cracking.
2. To determine the effect of various coarse aggregate gradations and sources of fine aggregate on freeze-thaw durability.
3. To monitor the performance of a test pavement with particular attention to subbase drainage facilities and various cement-aggregate combinations.

Due to the difficulties in moisture gauges and the lack of consistent trends, the moisture content studies were inconclusive. Further investigation of the effect of reduced particle sizes showed more evidence that a reduced particle size reduces the incidence of D-cracking susceptibility. This report also concluded that fine aggregate does little to influence D-cracking; the size of fine aggregate is too small to allow the pressure build-up to begin cracking of the aggregate and pavement. The monitoring of the test road at Vermilion primarily consisted of a visual inspection of the pavement slabs. Because the test pavement was only three years old at the time of inspection, no D-cracking was visible on the surface. This greatly limited the usefulness of this inspection data for the purposes of this study. No other new information was presented concerning the test pavement in this study.

The Significance of Pavement Design and Materials in D-Cracking, by David Stark (1986), was the last report on D-cracking prepared by Construction Technology Laboratories (with assistance from the Federal Highway Administration) for the Ohio Department of Transportation. This publication, considered an interim report, focused exclusively on the Vermilion test pavement. It restated the findings that reducing the maximum particle size minimizes or eliminates D-cracking. The report also noted that while a reduced coarse aggregate size lowered the incidence of D-cracking, it increased the incidence of intermediate transverse cracking. Ultimately, the BGSU study also found a clear relationship with reduced particle size and a reduction in the incidence of D-cracking for coarse aggregate from the same source, and a very clear indication that smaller coarse aggregate was linked to a higher incidence of intermediate transverse cracking. Some introductory findings about test road components were formulated in this report. Typical section, type of subbase, use of vapor barrier, and joint sealants appeared to have little bearing on the development of D-cracking. Though these findings concurred with the 1986 report in that typical section, type of subbase, and vapor barrier use were not significant in the development of D-cracking in the test road, this investigation found a significantly greater increase in D-cracking when neoprene sealant was used.

Further Portland Cement Studies

The next series of publications were also written by the Portland Cement Association.

The first two, Field and Laboratory Studies of the Effect of Subbase Type on the Development of D-Cracking, and Characteristics and Utilization of Coarse Aggregates Associated with D-Cracking were both written by David Stark (1970 and 1976 respectively), while the last publication, Effect of Maximum Size of Coarse Aggregate on D-Cracking in Concrete Pavements, was written by David Stark and Paul Klieger (1974). Though these documents are rather technical in nature, they pertain directly to many of the SR2 test road components and their effect on D-cracking. Field and Laboratory Studies of the Effect of Subbase Type on the Development of D-Cracking summarizes laboratory results found by the Portland Cement Association in their studies of subbase design and its influence on D-cracking. This study looked at a series of 4 foot by 5 foot by 6-inch thick concrete slabs containing coarse aggregates known to cause D-cracking that were cast on four different types of subbases.

These were:

1. Granular
2. Granular plus 4-mil polyethylene sheeting between the base and slab

3. Cement-treated

4. Clay

The concrete slabs also were made up of three different coarse aggregates, with varying degrees of D-cracking susceptibility. The slabs had moisture monitors placed in them, and the moisture levels at the bottom half of the slab (where D-cracking originates) was monitored over the course of 22 months. Unfortunately, no real conclusions could be drawn from the results, as there were no consistent trends in moisture levels for different subbases.

Characteristics and Utilization of Coarse Aggregates Associated with D-Cracking provided a summary of previous laboratory work of coarse aggregate done by the Portland Cement Association. The document discusses the two aspects most important to coarse aggregate weathering: moisture movements and critical saturation of the aggregate, and the response of the aggregate to cyclic freezing and thawing in concrete. This study goes into detail explaining coarse aggregate testing procedures, and determined that the pore structure of the coarse aggregate is the primary factor in determining D-cracking susceptibility. Further, it concludes that pavement design has little bearing on the degree of saturation of aggregates in pavement concrete. Lastly, the report found that the most feasible method of improving coarse aggregate durability is to reduce the maximum aggregate size of aggregates originating from quarries associated with D-cracking.

The last report, Effect of Maximum Size of Coarse Aggregate on D-Cracking in Concrete Pavements, also found that reducing the maximum particle size of coarse aggregates from sources associated with D-cracking to be beneficial. This conclusion was made after laboratory studies were undertaken to determine if freezing and thawing tests could substantiate the previous service records of coarse aggregate sources with respect to D-cracking. For this Vermilion project, coarse aggregates from 15 sources were used in a series of tests. These 15 coarse aggregates were divided into three different categories according to service record for coarse aggregate in 1/2 inch, 1 inch, and 1 1/2 inch maximum particle sizes. These categories were: 1) one for which D-cracking appears at the wearing surface in less than 8 years, 2) one for which D-cracking appears between 8 and 15 years, and 3) one for which D-cracking is not apparent in more than 15 years.

The 15 different coarse aggregate sources were tested according to the American Society for Testing and Materials (ASTM) procedure 666C. A failure criterion of 0.032 to 0.033 percent expansion in 350 or fewer freezing thawing cycles was established on the basis of the service records of 15 sources from which the test materials were obtained. The percent expansion of the coarse aggregates over time indicated lower quality coarse aggregates had much greater expansion after 350 cycles of freezing and thawing than higher quality aggregates. Also, the rates of expansion decreased significantly as the coarse aggregate size was reduced. This reduction was especially marked in the lower quality aggregates.

These findings are important, since expansion during freezing and thawing is the cause of D-cracking. In conclusion, the ASTM 666C test did succeed in distinguishing between coarse aggregates from sources associated with the development of D-cracking and those from sources with known satisfactory service records. Therefore, the report recommends the testing of coarse aggregate sources where D-cracking is a problem in order to determine the benefits to be derived from reducing the maximum particle size to improve durability. This study established the clear link between maximum size of the coarse aggregate and D-cracking in susceptible aggregates. For this reason, the SR2 test road included sections constructed from several gradations of a D-cracking susceptible aggregate. This allowed a practical comparison between sections of different gradations in field conditions.

Federal Transportation Research Board Studies

Transportation Research Record 853, Paxton (1982), a publication of the Federal Transportation Research Board contains several articles of relevance to the issue of components and their effect on D-cracking. The article D-Cracking: Pavement Design and Construction Variables, by Girard, Myers, Manchester and Trimm (1982), discusses some of the work done by Missouri in an effort to find the effect of construction variables on D-cracking. The variables studied included type, size, and characteristics of coarse aggregate, source of cement, design of concrete mix, and type of base. This study was conducted by surveying a sample of projects built in Missouri from 1927 to 1940 that showed joint distress. After analysis, fine aggregate and the brand of cement used did not appear to have any effect on D-cracking, nor did curing method. This finding agrees with Stark's (1991) final report and substantiates the decision to remove cement type and curing method from the SR2 test road statistical analysis.

Interestingly, weather conditions during construction were a factor, as projects completed in cooler temperatures had less deterioration. This is presumably due to stronger final strength in concrete with a slower cure time.

In the same document, it was reported that another Missouri study done in 1977 indicated pavements constructed with polyethylene moisture barriers exhibited D-cracking at an earlier age. Because of this, Missouri decided to eliminate use of the polyethylene sheeting moisture barrier in future Portland Cement Concrete (PCC) highway construction. These findings are in alignment with the findings in this study that found that pavements constructed with polyethylene moisture barriers were in slightly worse condition than those pavements not using a polyethylene moisture barrier. Another noteworthy observation from the Missouri study is the potential effectiveness of concrete curb-and-gutter designs with drop inlets and pipe drainage systems in reducing D-cracking. The ability of the drainage system to effectively carry the bulk of the surface water away from the pavement should reduce the subbase or base moisture conditions. Related to the Missouri study, Cedergren (1988) noted the importance of drainage to prevent D-cracking in his article Why All Important Pavements Should Be Well Drained. He mentions D-cracking progresses only in the presence of free water, and proposes the use of positive rapid drainage to reduce or eliminate D-cracking. Cedergren mentions high-permeability open-graded drainage layers as proposed in the Federal Highway Administration's Guidelines for the Design of Subsurface Drainage Systems for Highway Structural Sections, Cedergren, O'Brien and Arman (1972), as a remedy for D-cracking pavements.

Another field study was done by the state of Missouri in 1977. Twenty-eight projects, which had the 1-inch maximum-size coarse aggregate and polyethylene sheeting moisture barrier placed under all or part of the pavement, were included in this study. These projects were compared with 13 projects with 2-inch maximum-size coarse aggregate. The study investigated the effects within projects of polyethylene and between projects for effects of coarse-aggregate size. The conclusions reported at the termination of the study indicated that:

1. D-cracking increased slightly when a vapor barrier was used.
2. The use of the 1-inch maximum size aggregate increased D-cracking when used with frost-susceptible coarse aggregate.

Lastly, this study mentioned that many projects built in the Kansas City area in the 1950s, 1960s, and 1970s were constructed with a concrete curb-and-gutter design with a drop inlet and pipe drainage system. Observations of these pavements indicated little or no D-cracking. It was felt that the ability of the drainage system to carry the bulk of the surface water away from the pavement reduced the subbase moisture, and therefore reduced D-cracking.

The National Cooperative Highway Research Program Synthesis of Highway Practice published D-cracking of Concrete Pavements, Schwartz (1987), an assortment of various reports that synthesizes useful knowledge from all available sources. This report was the second Transportation Research Board report conducted on this subject. It described the mechanisms of D-cracking, summarized materials-acceptance and design techniques that can minimize D-cracking of new pavements, and described rehabilitation techniques for existing pavements. Because of the comprehensive manner of this report, it was very useful in understanding the framework of D-cracking. Chapter one introduced D-cracking, chapter two discussed the role of coarse aggregate in D-cracking concrete pavements, and chapter three covered preventing or minimizing D-cracking in concrete pavements.

Chapter one reviews factors that influence the development of D-cracking. Factors evaluated included environmental conditions, coarse aggregate, fine aggregate, cement, pavement design, subsurface drainage, and traffic. This topic is important to this research, since these same variables affect the amount of D-cracking in SR2 test road. The most significant of the environmental factors influencing the development of D-cracking are freezing temperatures and moisture, as D-cracking has been directly associated with the repeated cycles of freezing and thawing in the continuous presence of moisture. For this reason, the SR2 test road was placed in the northern part of the state, where more freezing and thawing cycles could influence the pavement.

Fine aggregate used in concrete pavements have been shown to have little influence on D-cracking. Since the SR2 test pavement uses one fine aggregate, fine aggregate can effectively be ruled out as a contributor to differing levels of D-cracking in the SR2 test pavement. This report found the possible influence of pavement design has not been clearly defined in the literature reviewed. However, some work related to this study found that pavements with a polyethylene moisture barrier displayed D-cracking at an earlier age than those pavements without the moisture barrier. This same conclusion will be found later upon this analysis of the SR2 test road.

Throughout most of all of the material referenced above a common thread of observations have been noted. These findings were 1.) How D-cracking is initiated through the synergistic affects of aggregate size, moisture, and freezing and thawing; 2.) How it can be controlled by roadbed design; 3.) The criticality of aggregate size; 4.) The origin of the aggregate and 5.) How D-cracking can be generally avoided. Each of these is discussed below.

D-Cracking Characteristics

D-cracking begins by the distress that develops in the coarse aggregate. As the overall crack pattern progresses laterally and upward through the slab, numerous other particles develop microcracks. These microcracks enlarge and extend into the surrounding mortar (Stark, 1974).

When D-cracking is observed in the above pattern, it is evidence that D-cracking was caused by the weathering of coarse aggregate saturated to a critical point. This interpretation is also supported by extensive research on D-cracking occurring in other areas of the midwest states and Canada. Deterioration of this type is initiated when atmospheric water penetrates open joints and cracks, and together with moisture already present beneath the pavement, raises the degree of saturation of the coarse aggregate to a critical level. During freezing, pressures generated in the aggregate through ice accretion then causes decay of the aggregate and surrounding mortar. As deterioration continues, cracks provide channels for migration of moisture into the slab and become additional sites for ice formation and the generation of excessive pressures operating to widen the existing cracks. If allowed to progress, the entire pavement slab will be destroyed.

Research has determined D-cracking can be controlled through proper selection of materials and may be in part controlled by various components of pavement design such as drainage and joint sealing. Avoiding coarse aggregates susceptible to freeze-thaw damage, prudent choice of subbase materials, and control of maximum coarse aggregate size has been found the most effective means of preventing D-cracking. Controlling the maximum size of coarse aggregate is the most important of these factors, as it has been found that reducing the maximum particle size can greatly reduce or eliminate D-cracking.

With this idea in mind, a critical particle size has been found, in which a particle above that size is not immune to the effects of freezing and thawing. If expansions exceed 0.032 percent expansion after 350 freeze-thaw cycles in accordance with ASTM 666C, the critical size is less than the maximum particle size being tested. If expansions are less, then the critical size is greater than the maximum particle size (Powers, 1955). In other words, in aggregates susceptible to freeze-thaw deterioration, reducing the size of the largest aggregates can delay or eliminate D-cracking, though reduction of the largest aggregate size will do little or nothing for aggregate durability in an aggregate not susceptible to decay.

The characteristics of aggregates associated with D-cracking have a number of characteristics in common. However, no single one can be used categorically to predict the inherent durability of the material (Verbeck, 1960). Of these, the characteristics most important are composition and response to freezing and thawing. Because of the lack of adequate identification and testing of absorption and pore characteristics, the intricacies of these laboratory procedures will not be further discussed, since no clear conclusions can be made.

Nearly all aggregates associated with D-cracking are of sedimentary origin. These range from pure limestone and dolomite to sandstone, graywackes, and other poorly consolidated materials. Interestingly, except for poorly consolidated materials, rock types over the same compositional ranges are known to have performed satisfactorily (Verbeck, 1960). Materials of igneous origin are not known to be associated with D-cracking.

Materials of metamorphic origin have given variable but generally satisfactory performance (Klieger, 1974).

Material from both crushed stone and gravel sources has been found to be associated with D-cracking. However, because of differences in the nature of the two types of deposits, greater variations have been observed in the performance of crushed stone materials in a given region. Ohio uses both sources, with more gravel being used in the southern part of the state, and more crushed stone in the north.

D-Cracking Avoidance

To avoid D-cracking, either the environment must be altered to prevent the aggregate from becoming critically saturated or the aggregate must be inherently durable. Observations indicate that environmental control is not presently sufficient to control D-cracking. As discussed earlier, D-cracking has been observed in a variety of pavements using measures such as vapor barriers, stabilized subbases, artificial drainage systems, and every other design variable. The only component of a highway system where D-cracking has not been observed is on bridge decks. Here, the concrete is not continuously exposed to 100% humidity, and therefore critical saturation is never accomplished (Klieger, 1974). Because it is currently not feasible to design a pavement that prevents an aggregate from becoming saturated, improving the durability of the aggregate is the most useful means of D-cracking prevention.

As previously mentioned, reducing the maximum particle size in the coarse aggregate can eliminate D-cracking. There are severe repercussions for doing so, however. In highway pavements, reducing the maximum size of aggregates may result in an increase in the number of intermediate transverse cracks that have significantly faulted. This is because of the reduced aggregate interlock and increased shrinkage, which occurs when using a reduced size aggregate. The expense of repairing the intermediate transverse cracks is a very important component of this research.

Summary of Issue

In summary, D-cracking is a complex issue, as each quarry has aggregates of differing quality, and in the case of gravel quarries, each individual aggregate particle has differing susceptibility to weathering. Pavement design through proper drainage may reduce the amount of water present in the pavement structure. Though more research is needed for definitive answers, it is well known that the presence of water and the subsequent freeze-thaw deterioration is the chief component of D-cracking deterioration. A critical size exists for aggregates of the same source, in which aggregates above that size are susceptible to freeze-thaw damage. Reducing the size of vulnerable aggregates below that critical size can eliminate D-cracking, but at a cost of increased transverse cracks.

CHAPTER III. ANALYSIS OF CONSTRUCTION VARIABLES

Procedures for SR 2, Vermilion Study

Most of the previous work on the investigation of the causes of D-cracking has been done in laboratory conditions. In order to evaluate factors thought to affect D-cracking that are not amenable to laboratory study, a test pavement was constructed in 1974 and 1975 by the Ohio Department of Transportation in Vermilion. This test pavement was conceived with two purposes in mind: first, to evaluate certain factors not amenable to laboratory study, such as drainage systems, joint sealant, etc.; and second, to confirm the importance of certain laboratory findings, such as aggregate particle size effects, and source of cement.

The test road location decision was based on several factors. The first factor was the climate of the region. Because D-cracking is caused by freezing and thawing, the northern part of Ohio provided a more appropriate test site than the southern portion. In addition, the proposed test site needed a terrain that had relatively uniform grade, and little variability in subgrade. After careful consideration, the State Route 2 (SR 2) bypass around Vermilion was chosen. The contract for construction was let in 1972, with the pavement construction taking place from 1974 to 1975.

When the statistical evaluation was done Dr. Nancy Boudreau of the Statistical Consulting Center at Bowling Green State University found a flaw in the statistical design of the initial construction of SR 2. The large number of variables that were included made it difficult to acquire an adequate number of unique sections for statistical design. Therefore many unique sections had to be combined or disregarded. For example, the cement source was disregarded and the various drainage sections had to be combined into three parameters. One of the biggest impediments was that the quality control aggregate, Woodville, only was used in the 1.5" size (not 0.5" and 1.0").

There were more unexpected problems with extracting the necessary data from the test pavement sections. In an attempt to find the amount of transverse cracking in comparison to the amount of joint D-cracking in the test pavement, a major problem surfaced. Documents that recorded the locations of joints that were damaged due to D-cracking, and transverse cracks before repairs to the highway were made in 1993 were missing from ODOT main offices. These documents were created before sections of the test pavement were replaced, and therefore would be especially valuable.

In response to the lack of consistent data concerning the test road joint and transverse crack locations, Professor Travis Chapin and John Dryden, BGSU Research Assistant, surveyed the entire test pavement in April of 1998. Both joint locations and repair locations were recorded, enabling a determination to the amounts of D-cracking that resulted in repair, and the amounts of mid-slab transverse cracking. Though many sections have been completely replaced, this information yielded the following for each section:

A. Joints replaced
B. Percentage of joints replaced per section
C. Mid-slab repairs
D. Percentage of mid-slab repaired per section

This data gave a good comparison (albeit non-statistical) of different amounts of pavement destruction that can be expected for an aggregate size and source.

Test Road Analysis

Over the next 18 years, several sections were overlaid with asphalt, for various reasons. In 1993, a major reconstruction of the pavement was done. The test sections were evaluated for damage, and any section more than 40% damaged was replaced entirely.

Fifty-Seven Sections Surveyed

In 1998, the 57 sections that were not overlaid or replaced were surveyed for purposes of this study. This survey measured:

1. The station locations of the joints.
2. The station locations of pavement repairs (both truck and passing lanes).
3. The length of pavement repairs (both truck and passing lanes).

As part of this study the resulting information was developed into the first of three groups of data that was used to assess the test pavement. A second grouping of data was extracted from the 1991 plans and from the project that overlaid the road with asphalt in 1996. Although joint locations were not known for the ten sections, it was possible using the plans to find the amount of pavement that was repaired before being overlaid. Lastly, the remaining 34 test sections were attributed with 100% pavement replaced. Unfortunately, this was the only way to account for the differences in deterioration, and was a compromise without better information being available. The decision was made to use 100% deterioration because the cost of using those components over the life of the pavement was most accurately described this way. For statistical evaluation, 99 sections were used in the evaluation of the total pavement quality while 57 of those sections were used in the evaluation of D-cracking of the joints and mid-slab cracking.

Selection of Composition Variables

After eliminating the Continuously Reinforced Concrete pavements not included in this study, the remaining sections are composed of some combination of six composition variables and eleven typical sections. The composition variables included:

- Three aggregate gradations
- Three sources of coarse aggregate
- Three uses of joint sealant
- Two different uses of a vapor barrier
- Three different portland cement sources
- Two different curing methods

The three gradations refer to the maximum size coarse aggregate used. The three maximum sizes used were 1.5 inch, 1 inch, and .5 inch. These maximum sizes correspond to AASHTO gradations of #4, #6, and #8, respectively.

The coarse aggregates originated from Woodville (Sy2), Canton (Sk 681), and Marion (Mn3). Of these, aggregate originating from Woodville was the highest quality of those three used in the test pavement. Previous testing and use in the state found it was not susceptible to D-cracking in any gradation. Canton coarse aggregate was the next best, and is susceptible to D-cracking only when used in the #4 gradation. Canton aggregate was the only gravel source used, while Woodville and Marion aggregates were crushed limestone. Marion coarse aggregate was the most common aggregate used, and was prone to D-cracking in all gradations.

Joints were sealed with neoprene, hot-poured bitumen, or were left entirely unsealed. The primary sealant used in most of the test pavement was hot-poured bitumen.

Polyethylene vapor barriers were used beneath the pavement slabs in some sections. These were placed between the pavement slabs and subbase, with the intent of preventing water from migrating into the concrete. Since the freeze-thaw action of water is the cause of D-cracking, the use of a vapor barrier would presumably prevent deterioration.

Typical Sections

There were a total of eleven different typical sections that were additional test pavement variables, differentiated by subbase. These are shown below in Figure 1.

Drainage/ Subbase	Material/ Number of Drains	Number of Test Sections
Drains	Granular (two drains)	
	Asphalt (two drains)	6
	Cement-treated (two drains)	6
	"Daylighted" granular	9
	Granular (one drain)	16
No drains	Granular (not drained)	24
	Asphalt (not drained)	7
	Cement-treated (not drained)	7
Clay	No subbase (asphalt shoulder, 15 inch slab)	2
	No subbase (concrete shoulder, 15 inch slab)	2
	No subbase (9 inch slab)	0

Figure 1. Typical test pavement sections showing draining, surface and number of sections of each type.

The differences among the typical sections are based on different drainage expectations. It was expected that the drained subbases would provide better pavement durability by conducting water away from the concrete, and those slabs laid directly on the clay subgrade without any drainage would provide the worst performance. Though the six sections laid directly on the subgrade did perform poorly in both the Stark report (1991) and in this study, the presence or absence of longitudinal drains in the shoulders appeared to have essentially no differential effect on the development of D-cracking.

Combining Categories

In order to clarify the results of the statistical analysis, it was necessary to combine the 11 different typical sections "subbase" into three groups: "drained," "not drained," and "clay." This enables a more accurate depiction of the effects of subbase drainage, by reducing the variability among the different composition variations in the statistical analysis. Among components, cement source and cure were not considered in this study. Cement was not felt to be a significant contributor to concrete life in Stark (1991) and was disregarded in this study's statistical analysis. Cure was also not considered a significant contributor in Stark's final report, and was also disregarded. By disregarding those factors, a better statistical analysis could be done. This allowed for the grouping of more observations under one analysis. Without these assumptions any statistical evaluation would be meaningless.

Measure of Distress

In the previous section, the various typical sections and components that were built into the test pavement were discussed. Also discussed was the removal of several components from statistical analysis, and the rationale behind their removal. To be able to measure the effect that these variables had on the test sections' deterioration, a methodology was established. The methodology needed to find:

1. The overall deterioration of pavement slabs.
2. The deterioration of the joints, presumably by D-cracking.
3. The severity of the faulting due to mid-slab cracking, presumably caused by poor aggregate interlocking.

It was decided that measuring square yards of pavement was the best method of determining the above requirements.

This became:

1. Percent of Bad Pavement (PBDPT).
2. The Percentage Bad Pavement at Joint (PBd-Jt).
3. The Percentage Joint Pavement that is Bad (PJPTBD).

These three descriptors of the pavement deterioration give a complete picture of where the deterioration occurred and how different materials combinations influenced the deterioration. In April of 1998, this information was acquired by walking the pavement and examining the 57 remaining sections that had not been overlaid or replaced. Each section was several hundred feet long.

Using these three descriptors, the data was entered into a SAS Institute Statistical Software program at the Bowling Green State University Statistical Consulting Center. Included in this data were the composition variables noted above: aggregate gradations, sealant, size, source and vapor barrier, and the number of pavement sections with each type of variable. These printouts are the result of statistical analysis accomplished on SR2, and are the basis for all statistical references on SR2 that follow. They are included in Appendix A for further reference and completeness.

The resulting statistical reports form the basis for the 17 graphics contained in Appendix B. "Percentages of Bad Pavement", "Bad Pavement at Joints" and "Joint Pavement that is Bad" are each further expanded into percentages by aggregate size, aggregate source, vapor barrier used, subbase and sealant used. Additionally, percent of slabs with mid-slab cracking and percentage of joints repaired are portrayed by a graphs. As each of the aforementioned areas is discussed below, the appropriate figure in Appendix B will be referenced.

Percent of Bad Pavement (PBDPT)

The "Percent Bad Pavement" is a descriptor that is most useful in determining the overall value of the different components in preventing damage to the entire pavement slab. For this discussion of PBDPT, 99 sections were studied. This included not only the 57 sections that were inventoried in April of 1998 but also 42 sections that had previously been overlaid or replaced. It is the percentage found by dividing the square yards of repaired or replaced pavement in each section by the number of square yards in a test section. The overall percentage of deterioration for all test sections was 43.9%. In other words, of 1000 square yards of test pavement, 440 square yards would be repaired or replaced after 19 years. It was found that aggregate source and subbase drainage were significant contributors to this deterioration (Figures A3 and A4), while the other factors of size, sealant, and vapor were not significant. Figures B1, B2, B3, B4, and B5 give graphical representations of this. This means that the only significant contributors to overall slab deterioration are the quality of aggregate used and the subbase drainage.

As mentioned earlier, the best aggregate source was from Woodville. Test sections that used this aggregate had only 12.4% deterioration, much lower than the overall average of 43.9%. This number is probably unduly influenced by the lack of use of 1 inch or .5 inch aggregates from Woodville. If these sizes were used, there most probably would have been mid-slab cracking, raising the overall deterioration percentage of this aggregate source.

The next best aggregate source was the Canton aggregate. When this aggregate source was used, 20.4% of the total square yards of these pavement sections were repaired. In other words, of 1000 square yards of test sections using Canton aggregate, 204 square yards were repaired or replaced after 19 years.

The worst performing aggregate was from Marion. 50.2% of all pavement sections that used Marion aggregate needed repair or replacement after 19 years.

Further statistical evaluation found certainty in the differences between the deterioration of sections using Marion aggregate and sections that used aggregate from Canton or Woodville. In other words, if the experiment was to be redone, there is certainty that sections using Marion aggregate would consistently be more deteriorated than sections using Canton/Woodville aggregate. However, statistical analysis cannot find with certainty significant differences in the deterioration of sections using Woodville aggregate (12.4%) and Canton aggregate (20.4%). Although Woodville aggregate has a lower percentage of deterioration than Canton aggregate in this study, statistical analysis shows these results are not certain, and may be different if the experiment was repeated.

There is some chance that a repeated experiment may find Canton aggregate may have less deterioration. This is a result of the relatively low number of sections constructed with these coarse aggregates. There were 12 sections for Canton, 7 for Woodville and 80 for Marion. This lack of certainty should be considered when interpreting the findings of this study.

Drainage. The other significant variable affecting overall pavement deterioration was subbase. Here, a significant difference was found between sections that were placed directly on the clay subgrade and those that used some kind of subbase. This significance arose because all sections laid directly on clay were replaced, and thus were credited with 100% deterioration. No significant difference was found between pavements that were drained and those that were not drained. There were 58 sections that were "drained" showing 44.1% deterioration. There were 36 sections that were "not drained" showing 35.9% deterioration (Figure A3). Though it is not statistically significant "drained" performed almost the same as "not drained" (Figure B2, B10 and B14). It is important to note that conventional wisdom is that "drained" pavements should perform better than "not drained". They did not. In fact they were opposite to conventional wisdom. This concurs with the findings of Stark (1991) that reported subbase had no discernable effects on D-cracking of the test pavement through 1990. In addition, type of subbase was found to have on significant effect on the occurrence of D-cracking as reported by Traylor (1982) in his Efforts to Eliminate D-cracking in Illinois.

Percentage Bad Pavement at Joint (PBd-Jt)

This category is a descriptor of where deterioration occurred at the joint in the different test pavements. The pavement studied consisted of the 57 sections surveyed in the April of 1998. Percentages were calculated by dividing the square yards of joint pavement considered defective by the total square yards of bad pavement. Overall, this statistical analysis found the "Percentage Bad Pavement at Joint" for the entire test pavement surveyed (not including sections replaced or paved over) to be 28.4%. The 28.4% of the total pavement deterioration occurred in the "joint area," that is, within 3 feet of the joint on either side. The remaining 72% of the deterioration was elsewhere.

Figures A5 and A6 give the statistical evaluation of the variables for "Bad Pavement at Joint (PBd-Jt)." Of these variables, only size and sealant are significant. Figure B6 shows that the percentage of deterioration at the joint increased as aggregate size increased. Conversely, it is was found that a small aggregate lowered the percentage, as the poorer aggregate interlock that goes along with the use of smaller aggregate is thought to result in more mid-slab cracking. Further, it was found that the use of differing joint sealants (Figure B7) had an impact on the amount of "Percent Bad Pavement at Joint." Neoprene performed the worst.

Aggregate. Using .5-inch maximum aggregate reduced the amount of deterioration occurring at the joint area but increased the amount of deterioration occurring at the mid-slab. The percentage of deterioration occurring at the joint when using .5-inch aggregate is 13.9%, half of the amount found for the overall pavement at 28.4% (Figure A5 and A6). The results for pavement sections using 1-inch aggregate were almost identical to sections using .5-inch aggregates, or 13.6% deterioration occurring at the joint. This statistic is probably unduly influenced however; a much higher percentage of good-quality aggregate was used in the test sections using 1-inch aggregates than in the sections using .5-inch or 1.5-inch aggregates. The average percentage for sections using 1.5-inch aggregate is higher than average, at 34.3%. This reflects the greater D-cracking at the joint, and presumably the greater aggregate interlock achieved that occurs with a larger aggregate (Figure B6). Further statistical testing reveals a significant difference in the expected deterioration between .5-inch and 1.5-inch aggregates. In other words, if this experiment was repeated, there is certainty that 1.5-inch aggregate would consistently perform worse than .5-inch aggregate. There was not enough of a distinction between .5-inch and 1-inch aggregate, or 1-inch and 1.5-inch aggregate to be conclusive in their predictive power of deterioration. At the joints this is partly caused by the relatively small number of 1-inch aggregate sections in the test pavement.

Sealant. The other significant influence of "Percent Bad Pavement at Joint" was the type of sealant system used. Overall, using no sealant at all in the joints had the lowest percentage of deterioration occurring at the joint area, only 23.2%. This was followed by hot-pour sealant with 24.5%, and neoprene sealant had 60.5% deterioration (Figure A5 and Figure B7). Further statistical analysis shows there is no significant difference between using hot-pour sealant and no sealant at all. Though regarded as being statistically significant, it must be pointed out that there were 45 sections for the hot pour and only 6 sections for neoprene and 6 for none. Furthermore, because it of the limited number of sections it was not possible to factor the aggregate quality into this comparison.

Source. Source is expected to be another determinant. As aggregate quality increases, the reduced amount of D-cracking should lower this metric (Figure A5 and B8). In this study, source did not significantly contribute to the "Percentage Bad Pavement at Joint." The better quality aggregates did perform better in reducing repairs in the joint area, but not to enough of a degree to be found a significant contributor. One reason for this is the distribution of test sections using Woodville aggregate. Fully half of all test sections using Woodville aggregate has an identical component composition (Hot-pour sealant, drained subbase, no vapor barrier). Furthermore, Woodville aggregate was used only in the 1.5-inch gradation, and this caused difficulty in comparing sources. As mentioned above, source was not a significant factor to D-cracking in this category. As depicted in Figures B9 and B10, neither were vapor barriers or subbase.

Percent Joint Pavement Bad (PJPTBD)

This number pertains to another determinant of the joint deterioration. The percentage was calculated by finding the amount of joint pavement (area within 3 feet of either side of a joint) that was repaired, then dividing that number by the number of square yards of joint pavement. For example, if a test section had eight joints, it would have 64 square yards (2 yards long by 4 yards wide times 8 joints) of joint pavement. If four of those joints had been repaired (32 square yards), then the "Percent Joint Pavement Bad" would be 50%. Overall, 28% of all joints in the test section surveyed were replaced at 19 years. For the same reasons, size and sealant were expected to influence "Percentage Bad Pavement at Joint," they were also expected to be determinants of the percentage of joints deteriorated. Moreover, source was also expected to be a determinant, because better quality aggregate is expected to reduce D-cracking, directly influencing joint deterioration.

These expectations corresponded with the findings of the statistical analysis. Source, size, and sealant (Figures B11, B12 and B13) were all significant influences of joint deterioration, most probably caused by D-cracking. The better sources reduced the amount of D-cracking, as did the use of smaller aggregate sizes. Using no sealant or hot-poured sealant reduced the amount of D-cracking in comparison to those sections that used neoprene strips. Figure A7 gives the statistical data.

Source. Source was a definitive factor in reducing joint damage (Figure B11). Test sections using aggregate from Woodville had only 9.0% of joints repaired, in comparison to 28.2% overall. Canton followed with 15.6%, and Marion was the worst at 34.8%. Subsequent testing showed Marion aggregate would consistently remain worse than both Canton and Woodville aggregates, but Canton and Woodville aggregates could not confidently be considered different from each other.

Aggregate Size. Sections using .5-inch aggregate had the lowest joint damage, at 12.6% versus 28.2% percent being the average across all sections (Figure B12). Sections using 1-inch aggregate had the next lowest joint damage, with 20.0% of all joints in these test sections needing repair after 19 years. Lastly, those sections using 1.5-inch aggregate had 33.8% of their joints damaged. As with "Percentage Bad Pavement at Joint," further statistical testing showed pavement sections built with .5-inch aggregate was conclusively better than those built with 1.5-inch aggregate, but neither .5-inch aggregate sections or 1.5-inch aggregate sections are significantly different than sections built with 1-inch aggregates. This may be because of the relatively small number (7) of sections built with 1- inch aggregate.

Sealant. Lastly, the use of sealant had effects on determining joint damage that were very similar to its affect on "Percentage Bad Pavement at Joint" (Figure B13). Joints using neoprene sealant had an average of 56.7% damage, twice the average percentage. 25.1% of joints using hot-pour sealant sustained damage, the same as joints using no sealant at all that was 24.6%. However, it must be pointed out that because it of the limited number of sections it was not possible to factor the aggregate quality into these comparisons.

In this category of the study the effects of the subbase and vapor barrier were not found to be significant (see Figures B14 and B15).

An Argument for Using Small Aggregate

Though aggregate size was not a determining factor of total quantity of bad pavement (Figures A3 and A4), an argument can be made that under today's design criteria the use of small aggregate can be beneficial. The joint spacing is no longer 40 feet as on the Vermilion project. The present ODOT design criterion is 15 feet for plain concrete. If we view the information gathered from the Vermilion project with the knowledge of the reduced slab length, the use of smaller aggregate size could be beneficial.

One of the disadvantages of the smaller aggregate was its susceptibility to mid-slab cracking and thus resulting in a higher quantity of overall deterioration. The following is a tabulation of the various deterioration percentages as presented in this report (Figures A3, A5, and A7):

Size	PBDPT	PBd-Jt	PJPTBD
0.5"	49.8%	13.9%	12.6%
1.0"	16.9%	13.6%	20.0%
1.5"	45.4%	34.3%	33.8%

If the above tabulation is viewed in the context of the influence that the 40-foot slabs and mid-slab cracking had on these percentage, it can be assumed that the 49.8% for the deterioration of the overall pavement would have been reduced. In other words, if the Vermilion job had had 20-foot slabs as in today's design criteria then the 0.5" aggregate size would have clearly been the better performer. But it must also be remembered that the loser, i.e., the 1.5" size, was highly influenced by the fact that it was a known D-cracking susceptible aggregate.

Interactions

In the statistical analysis, a check was made for the possible combined effects of two or three components acting together, influencing pavement deterioration. For example, this check tested the possibility that sealant, working in combination with subbase, had a positive or negative effect. An analogy from medicine is the effect of sleeping pills taken with alcohol has an interactive effect; taking the two together has a different effect than taking one or the other by themselves.

Though some combinations had interactions, contemporary guidelines used by the statistical consultants who performed the analysis did not allow any of the combinations to be considered. Because of this, only the main effects (the original components) could be considered. No combinations of components can be used in a scientifically valid study. The overall strength of the statistical model was not strong enough. However, for informational purposes, graphs that show the effects of combining aggregate size and aggregate source are shown as Figures B16 and B17. Figure B16 compares aggregate size/source with the percentage of slabs constructed with that aggregate that had at least one mid-slab crack. Figure B17 compares aggregate size/source with the percentage of joints constructed with that aggregate which were damaged. The graphs clearly show that joint damage (presumably D-cracking) increases as aggregate size increases, while mid-slab cracking decreases as aggregate size increases.

Cost Analysis

With the reality that present-day design criteria is a shorter joint spacing, the only variables to analyze are the aggregate size and quality. If the argument were being made to consider 15-foot versus 40-foot joint spacing then an analysis of the cost of additional dowel assemblies would be warranted. But that is not the case. Furthermore, wire mesh reinforcing is not a consideration anymore nor could it be addressed in this study because no applicable plain concrete was used.

Aggregate size and quality. The following gives a broad-brush look at the value of having the right aggregate for the job. A mile of two-lane concrete pavement requires about 3500 tons of coarse aggregate. If in order to get the "right" aggregate an additional $4 per cubic yard must be spent, the resulting increase in the pavement cost is $14,000. In this study it was shown that using the best aggregate (Woodville) resulted in a pavement deterioration of just 12.4% over a 23-year life. By using a poor aggregate (Marion), this percentage jumped to 50.2% (much of the Marion aggregate pavement didn't last 19 years). This is an increase of 37.8%. If a cost of $50 per square yard is applied to the repair of this 37.8% for a mile of two-lane concrete pavement, it amounts to about $250,000. Even if $14,000 is inflated at 5% per year, it only amounts to $43,000. This is nothing compared to the repair cost of $250,000.

Regardless of what it costs (within reason), it is imperative that the correct aggregate is used.

Aggregate Manufacturers' Survey

Appendix C shows a sample of a questionnaire that was sent out to 150 aggregate producers throughout Ohio. The list was acquired from the ODOT Materials Lab. One of the concerns of ODOT officials (and any other specifying agency) is that a new specifications requirement may unduly increase the cost of the specified product without proportionally increasing the value of the finished product. The note, 451-97, requires that only #57 aggregate that has passed ASTM 666, freeze-thaw testing, be used on projects that have more than 10,000 SYS. Before this note came into effect, one option was to use the smaller #8 aggregate when D-cracking susceptible aggregates are encountered. Because of their smaller size, #8's are not typically susceptible to D-cracking. Though the use of #8's may seem to be a logical solution, ODOT feels that #8's are more susceptible to mid-slab cracking.

The purpose of this section is not to discuss the merits of note 451-97 but to determine whether the implementation of this note makes any difference in the final cost to ODOT for the construction of concrete pavements.

The questionnaire was sent on May 22, 1998 to 150 aggregate producers. The response rate was very low. Only eighteen responses were received for a 12% return rate. One of the major problems seemed to be accuracy of the addresses and contact individuals at the various quarries. The Post Office returned 23 to us as being undeliverable.

There were eleven questions to the questionnaire. The following is a tabulation and discussion of the responses.

1) Are you aware of Proposal Note 451-97?

Yes, very knowledgeable	2 responses
Yes, knowledgeable	4 responses
Yes, somewhat knowledgeable	7 responses
No, have not heard of it	5 responses

Discussion: The rationale for this question was to see if this proposal note is an issue with the aggregate industry. Most quarries have heard about the note but they don't seem to have had much experience with it (May of 1998 but awareness has increased).

2) Do you think that the new specifications will have an impact on your aggregate operations?

Yes, a significant positive impact	2 responses
Yes, a slightly positive impact	6 responses
No, impact at all	3 responses
Yes, a slightly negative impact	1 response
Yes, a significant negative impact	0 responses
Don't know	5 responses

Discussion: Of those quarries that have formed an opinion it seems to be positive. However, there seems to an uncertainty about the impact.

3) In general, does your aggregate pass the ASTM 666 freeze-thaw durability test?

Always passes	6 responses
Passes most of the time	4 responses
Rarely passes	0 responses
Never passes	1 responses
Our quarry has not been tested	5 responses
Don't know	4 responses

Discussion: This question was asked because the 1997 proposal note virtually eliminated the use of any coarse aggregate that hasn't passed this durability test. Though there were more positive responses than there were negative, it appears that many sources have not been tested.

4) Because # 8 aggregate is smaller than #4 and #57 aggregate, it would be logical that #8 aggregate is more expensive than #4 and #57 aggregate to produce.

Is this true?

Almost always true	7 responses
Sometimes true	7 responses
Almost never true	3 responses

Discussion: The rationale for this question was to gain an understanding of whether or not #8's are inherently more expensive to produce. We discovered a problem with this question after it was sent out. It was pointed out in a phone conversation with one of the quarries that this would be answered differently by a gravel producer than a crushed stone producer. In gravel operations the #8 size already exists in nature and it's just a matter of screening it off. In a crushed stone operation, the #8 size has to be made with the crushing operation. From the responses and other phone conversations, it seems that the #8 would take more crushing (and possible expense) in the crushed stone operation.

5) Because #8 aggregate is smaller than #4 and #57 aggregates, it would be logical that the production # 8 aggregate yields significantly <u>more</u> waste fines.

Has this been your experience?

Yes, significantly more waste fines	8 responses
No, not significantly more waste fines	5 responses
No effect at all	3 responses

Discussion: Once again the response to this question depends on the whether it is a gravel operation or a crushed stone operation.

6) The availability of # 8 aggregate vs. #4 aggregate vs. #57 aggregate is almost always a factor of market demand.

Almost always	10 responses
Sometimes	7 responses
Almost never	0 responses

Discussion: Market demand does have an influence on the availability of the various aggregate sizes.

7) In an average year (over the past three years), what has been the cash price ($/ton) at your plant picked up by the customer for:

Quarry Sources
(1998 Prices)

	1	2	3	4	5	6	7	8	9	10
#4	5.25	6.00	3.50	-	5.00	5.80	6.20	5.50	8.50	same
#57	6.25	6.00	3.75	6.90	5.75	5.90	6.20	5.60	8.50	same
#6	6.25	6.00	3.50	-	5.75	5.90	-	5.60	8.50	-
#8	5.25	6.25	4.00	5.40	5.25	6.00	6.45	5.75	8.50	same

Quarry Sources

	11	12	13	14	15	16	17
#4	4.85	5.00	5.25	4.75	7.75	4.60	4.25
#57	4.85	6.00	5.75	5.40	7.75	5.00	4.50
#6	-	6.10	5.75	5.80	N/A	5.00	4.50
#8	5.25	6.10	5.50	5.60	7.75	5.35	4.75

Discussion: The basic focus in this study was to do a comparison of the large aggregate sizes (#4, #57, and #6) vs. the small aggregate size (#8). Does it make any difference in the cost of the completed concrete pavement whether a large aggregate was used or a smaller aggregate? If the average prices tabulated on the above chart for the large aggregate sizes (#4, #57, and #6) are compared to the average price for the small aggregate (#8), the difference is that the small aggregate is only $0.16/ton more expensive than a large aggregate. For a nine-inch pavement this would equate to only four or five cents per square yard. This is difference is insignificant. Furthermore, it is a conclusion, which would help support the change towards the larger aggregates.

8) If market demand were not a factor, which aggregate gradation would you prefer to produce? (Check one)

#4 aggregate	6 responses
#57 aggregate	6 responses
# 6 aggregate	1 response
# 8 aggregate	3 responses

Discussion: The larger aggregates seem to be cheaper to produce. For a crushed stone operation this would seem to be logical. Three responded that it made no difference.

9) Does it make any difference from a production standpoint (with no consideration of market demand) whether you produce #4, #57, #6, or #8 aggregate?

No	6 responses
Yes	9 responses

Discussion: Most plants have an optimum gradation combination but others seem to be able to respond to market demands.

10) Based on 100 percent, what is the ratio of #4, #57, #6, #8 coarse aggregate that your plant can produce at maximum capacity?

Quarry Source:

	1	2	3	4	5	6	7
#4	30	15	8	-	20	25	10
#57	25	23	32	40	25	25	20
#6	-	-	-	-	-	-	10
#8	20	17	16	60	30	15	25
#9	10	5	-	-	-	-	-
304	-	40	44	-	-	-	45
sand	15	-	--	-	1 -	35	-

Quarry Source:

	8	9	10	11	12	13	14
#4	10	15	30	-	-	15	-
#57	20	25	25	39	-	25	10
#6	-	18	10	-	-	-	25
#8	18	32	15	26	-	22	20
#9	-	-	-	-	-	-	-
304	52	-	-	-	-	-	30
sand	-	32	-	35	66	26	15

Discussion: There doesn't appear to be a consensus of what the optimum ratio of aggregate sizes is. If there had been a consensus that one aggregate size was optimal then it could be deducted that requiring that size in the specification would produce a pavement, which would be less expensive. This does not seem to be the case.

11) What percentage of your aggregate is sold to ODOT projects?

10 11 5 33 20 35 25 33 10 20 0 10 15 25 15

Discussion: The average market share that ODOT has of these plants is 18%. Though important there would not seem to be such a large amount of ODOT work that a statewide implementation of Proposal Note 451-97 would significantly increase the cost to ODOT. The number of acceptable plants would be reduced and may result in higher aggregate prices from those plants that are able to meet the more restrictive specification.

Questionnaire Summary

As stated before, one of the concerns of ODOT officials (and any other specifying agency) is that a certain requirement to use only #57 aggregate that has passed the freeze-thaw test will unduly increase the cost of the completed concrete pavement without proportionally increasing the value of the finished product. The findings of the questionnaire give little indication that forcing the use of quality #57's will significantly increase in the cost of the product. The reported prices FOB at the plant for #57's vs. #8's actually argue slightly in favor ($0.16/ton cheaper) of the use of #57's. There doesn't seem to be anything that is inherent in the aggregate production process that would indicate that a problem would develop. The increase involved in not using local #8's would be $0.80 per square yard. For a mile of two-lane highway that equates to about

$12,000 in increased construction costs. If the increased quality of the #57's used can decrease by one per cent the surface area to be repaired then the use of the quality #57's is worth the initial expense.

Missing from this report is the cost of shipping quality aggregates into an area that does not have naturally occurring quality aggregate. The study did not investigate this aspect. It is beyond the scope of the project. To do this correctly, a study would have to be made of actual quotations for jobs that have been bid since the implementation of Note 451-97.

Findings

It is important to note that these findings should not be applied to concrete pavements in general. Two major criteria that greatly influence these findings are that the major portion of the aggregate used was from a known D-cracking susceptible source (Marion) and that the joint spacing was at 40 feet. The Vermilion test pavement did provide a verification of useful information regarding the contributions different variables make towards maintaining the integrity of both joints and pavement. Most importantly, the study found:

- Aggregate size was not a determining factor of total quantity of bad pavement. With small aggregate the problem was mid-slab cracking. With large aggregate the problem is joint deterioration due to D-cracking (Figure A3 and B3). Figures A4, A6, and A8 give an explanation of statistical significance.
- Aggregate source was a determining factor of the total quantity of bad pavement (Figure A3 and B1). Figure A4 gives an explanation of statistical significance.

- To reduce D-cracking, a high quality and/or small sized coarse aggregate should be used. This can be inferred by looking at Figure B11 and B12.
- To reduce mid-slab cracking, larger aggregate sizes should be used. This can be inferred by looking at Figure B16. Mid-slab cracking was expected as part of the Vermilion design of the 1970s.
- Use of vapor barriers does not increase pavement performance (Figure B5).
- Subbases with drains performed about the same as ones without drainage (Figures A3 and B2). Figure A4 gives an explanation of statistical significance.
- The use of no joint sealant prevented joint deterioration due to D-cracking as well or better than hot pour or neoprene. Neoprene was significantly worse (Figures A5 and B7). Figure A6 gives an explanation of statistical significance.
- Pavements should not be placed directly on subgrade (Figure A3 and B2).

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APPENDIX A

STATISTICAL ANALYSIS REPORTS

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General Linear Models Procedure
Class Level Information

Class	Levels	Values
SEALANT	3	Hot Pour Neoprene None
SIZE	3	.5 in. 1 in. 1.5 in.
SOURCE	3	Canton Marion Woodville
SUBBASE1	2	Drained Not Drained
VAPOR	2	None Poly

Number of observations in data set = 57

A1 Variables Chosen for Analysis (57 Observations)

"**Class**" indicates the five different variables chose for analysis. "**Levels**" is the number of different components in each variable. "**Values**" are the names of each component in each variable. "**Number of observations in data set**" indicates the number of different pavement sections analyzed. "**57**" is the number of sections which were surveyed in April 1998.

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General Linear Models Procedure
Class Level Information

Class	Levels	Values
SEALANT	3	Hot Pour Neoprene None
SIZE	3	.5 in. 1 in. 1.5 in.
SOURCE	3	Canton Marion Woodville
SUBBASE1	3	Clay Drained Not Drained
VAPOR	2	None Poly

Number of observations in data set = 99

A2 Variables Chosen for Analysis (99 Observations)

"**Class**" indicates the five different variables chosen for analysis. "**Levels**" is the number of different components in each variable. "**Values**" are the names of each component in each variable. "**Number of observations in data set**" indicates the number of different pavement sections analyzed. "**99**" is the number of sections.

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General Linear Models Procedure

Level of SIZE	N	PBDPT Mean	PBDPT SD
.5 in.	18	49.8333333	41.5157660
1 in.	8	16.8750000	8.3398784
1.5 in.	73	45.4520548	41.3481294

Level of SOURCE	N	PBDPT Mean	PBDPT SD
Canton	12	20.4166667	26.0295578
Marion	80	50.2250000	41.3692771
Woodville	7	12.4285714	4.6496288

Level of SUBBASE1	N	PBDPT Mean	PBDPT SD
Clay	5	100.0000000	0.0000000
Drained	58	44.0862069	41.3285019
Not Drained	36	35.9166667	35.5144840

Level of VAPOR	N	PBDPT Mean	PBDPT SD
None	90	44.9111111	40.6781405
Poly	9	34.2222222	37.6356805

Level of SEALANT	N	PBDPT Mean	PBDPT SD
Hot Pour	75	41.7866667	39.5905713
Neoprene	10	40.6000000	41.2315952
None	14	57.8571429	43.9192566

A3 Mean and Standard Deviation for Variables of Percent of Bad Pavement (PBDPT)

The column "**N**" indicates the number of sections for each component of each variable. The number of sections of pavement using .5 in. coarse aggregate is 18.

The column "**Mean**" shows the average amount of bad pavement for all sections containing that component. The mean amount of bad pavement for sections using .5 in. coarse aggregate is 49.833 percent.

The column "**SD**" is the standard deviation of the average amount of bad pavement for all sections using that component. The standard deviation for the amount of bad pavement for sections using .5in. coarse aggregate is 41.515.

General Linear Models Procedure

Dependent Variable: PBDPT Pct Bad Pavement

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	9	42975.42529554	4775.04725506	3.65	0.0006
Error	89	116560.21106809	1309.66529290		
Corrected Total	98	159535.63636364			

R-Square	C.V.	Root MSE	PBDPT Mean
0.269378	82.36185	36.18929804	43.93939394

Source	DF	Type I SS	Mean Square	F Value	Pr > F
SOURCE	2	16751.05541126	8375.52770563	6.40	0.0025
SUBBASE1	2	14136.25854837	7068.12927419	5.40	0.0061
SIZE	2	5155.90140176	2577.95070088	1.97	0.1457
SEALANT	2	3821.94559019	1910.97279510	1.46	0.2379
VAPOR	1	3110.26434396	3110.26434396	2.37	0.1269

Source	DF	Type III SS	Mean Square	F Value	Pr > F
SOURCE	2	14895.01254153	7447.50627077	5.69	0.0047
SUBBASE1	2	12630.04399262	6315.02199631	4.82	0.0103
SIZE	2	5034.96244512	2517.48122256	1.92	0.1523
SEALANT	2	4741.54941848	2370.77470924	1.81	0.1696
VAPOR	1	3110.26434396	3110.26434396	2.37	0.1269

A4 Probability of Validity for Percent of Bad Pavement (PBDPT).

Dependent variable, "**PBDPT**" is the variable analyzed, the percent bad pavement of each section. The "**Pr > F**" of 0.0006 shows the probability of validity; since any value under .05 is accepted as valid by current statistical procedures, this overall model is considered valid. The "**R-square**" of 0.269 reports the strength of the model; 26.9% of the percent bad pavement is predicted with this model. The "**PBDPT Mean**" is the percent bad pavement found for all pavement sections.

The "**Pr > F**" values for **Source** (0.0047) and **Subbase** (0.0103) indicate these individual variables do have a definite effect on the percent of bad pavement in a section. The "**Pr > F**" values for **Size** (0.152), **Sealant** (0.696), and **Vapor** (0.126) indicate these variables are not significant, and do not conclusively affect the percent bad pavement of pavement sections.

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General Linear Models Procedure

Level of SIZE	N	PBD_JT Mean	PBD_JT SD
.5 in.	10	13.9000000	21.7227479
1 in.	7	13.5714286	14.7293035
1.5 in.	40	34.3000000	29.4654948

Level of SOURCE	N	PBD_JT Mean	PBD_JT SD
Canton	11	24.7272727	29.9201969
Marion	40	31.2000000	28.3197856
Woodville	6	14.3333333	22.6421436

Level of SUBBASE1	N	PBD_JT Mean	PBD_JT SD
Drained	29	22.8275862	28.7527253
Not Drained	28	33.7142857	26.9455654

Level of VAPOR	N	PBD_JT Mean	PBD_JT SD
None	50	27.8000000	29.0200494
Poly	7	30.8571429	22.7920203

Level of SEALANT	N	PBD_JT Mean	PBD_JT SD
Hot Pour	45	24.5333333	25.1537093
Neoprene	6	60.5000000	29.3308711
None	6	23.1666667	32.5048714

A5 Mean and Standard Deviation for Variables of Percent for Bad Pavement at Joint (PBd-Jt)

The column "**N**" indicates the number of sections for each component of each variable. The number of sections of pavement using .5 in. coarse aggregate is 10.

The column "**Mean**" shows the average amount of bad pavement occurring at the joint for all sections containing that component. The mean amount of bad pavement occurring at the joints for sections using .5 in. coarse aggregate is 13.900 percent.

The column "**SD**" is the standard deviation of the average amount of bad pavement for all sections using that component. The standard deviation for the amount of bad pavement occurring at the joints for sections using .5 in. coarse aggregate is 21.722.

General Linear Models Procedure

Dependent Variable: PBD_JT Pct Bad Joint

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	8	14437.02333990	1804.62791749	2.89	0.0104
Error	48	30003.22227414	625.06713071		
Corrected Total	56	44440.24561404			

R-Square	C.V.	Root MSE	PBD_JT Mean
0.324864	88.73453	25.00134258	28.17543860

Source	DF	Type I SS	Mean Square	F Value	Pr > F
SOURCE	2	1646.33046252	823.16523126	1.32	0.2775
SUBBASE1	1	1453.73066876	1453.73066876	2.33	0.1338
SIZE	2	5775.16794925	2887.58397462	4.62	0.0146
SEALANT	2	5452.09618316	2726.04809158	4.36	0.0182
VAPOR	1	109.69807621	109.69807621	0.18	0.6771

Source	DF	Type III SS	Mean Square	F Value	Pr > F
SOURCE	2	1242.58193206	621.29096603	0.99	0.3776
SUBBASE1	1	1418.14921014	1418.14921014	2.27	0.1386
SIZE	2	4445.97972356	2222.98986178	3.56	0.0363
SEALANT	2	5340.42256118	2670.21128059	4.27	0.0196
VAPOR	1	109.69807621	109.69807621	0.18	0.6771

A6 Probability of Validity for Percent of Bad Pavement at Joint (PBd-Jt).

The "**Dependent Variable**," "**PBD_JT**" is the variable analyzed, the amount of bad pavement of each section occurring at the joint. The "**Pr > F**" of 0.010 shows the probability of validity; since any value under .05 is accepted as valid by current statistical procedures, this overall model is considered valid. The "**R-square**" of 0.324 reports the strength of the model; 32.4% of the percent bad pavement is predicted with this model. The "**PBD_JT Mean**" is the average percent bad pavement occurring at the joints of all pavement sections.

The "**Pr > F**" values for **Size** (0.036) and **Sealant** (0.019) indicate these individual variables do have a definite effect on the percent of bad pavement in a section. The "**Pr > F**" values for **Source** (0.277), **Subbase** (0.133), and **Vapor** (0.677) indicate these variables are not significant, and do not conclusively affect the percent bad pavement occurring at the joints.

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General Linear Models Procedure

Level of SIZE	N	PJTPTBD Mean	PJTPTBD SD
.5 in.	10	12.6000000	18.0690035
1 in.	7	20.0000000	25.7487864
1.5 in.	40	33.8250000	26.5522103

Level of SOURCE	N	PJTPTBD Mean	PJTPTBD SD
Canton	11	15.6363636	14.4310272
Marion	40	34.8250000	27.4962818
Woodville	6	9.0000000	16.1864141

Level of SUBBASE1	N	PJTPTBD Mean	PJTPTBD SD
Drained	29	23.0000000	27.0594583
Not Drained	28	34.0000000	24.6110484

Level of VAPOR	N	PJTPTBD Mean	PJTPTBD SD
None	50	27.3000000	26.1301687
Poly	7	36.2857143	27.7711738

Level of SEALANT	N	PJTPTBD Mean	PJTPTBD SD
Hot Pour	45	25.1333333	24.1233572
Neoprene	6	56.6666667	18.5759701
None	6	24.6666667	34.3258892

A7 Mean and Standard Deviation for Variables of percent of Joint Pavement that is Bad (PJPTBD)

The column "**N**" indicates the number of sections for each component of each variable. The number of sections of pavement using .5 in. coarse aggregate is 10.

The column "**Mean**" shows the average amount of bad pavement occurring at the joint for all sections containing that component. The mean amount of bad pavement occurring at the joints for sections using .5 in. coarse aggregate is 12.600 percent.

The column "**SD**" is the standard deviation of the average amount of bad pavement for all sections using that component. The standard deviation for the amount of bad pavement occurring at the joints for sections using .5 in. coarse aggregate is 18.069.

General Linear Models Procedure

Dependent Variable: PJTPTBD Pct Joint Bad Pavement

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	8	15489.67809958	1936.20976245	4.03	0.0010
Error	48	23090.04119867	481.04252497		
Corrected Total	56	38579.71929825			

	R-Square	C.V.	Root MSE	PJTPTBD Mean
	0.401498	77.21821	21.93268166	28.40350877

Source	DF	Type I SS	Mean Square	F Value	Pr > F
SOURCE	2	5701.39884370	2850.69942185	5.93	0.0050
SUBBASE1	1	1468.24857955	1468.24857955	3.05	0.0870
SIZE	2	5163.08381626	2581.54190813	5.37	0.0079
SEALANT	2	3145.51728210	1572.75864105	3.27	0.0466
VAPOR	1	11.42957796	11.42957796	0.02	0.8781

Source	DF	Type III SS	Mean Square	F Value	Pr > F
SOURCE	2	4301.11441695	2150.55720847	4.47	0.0166
SUBBASE1	1	1280.50992111	1280.50992111	2.66	0.1093
SIZE	2	4345.17871719	2172.58935859	4.52	0.0160
SEALANT	2	3104.49799244	1552.24899622	3.23	0.0484
VAPOR	1	11.42957796	11.42957796	0.02	0.8781

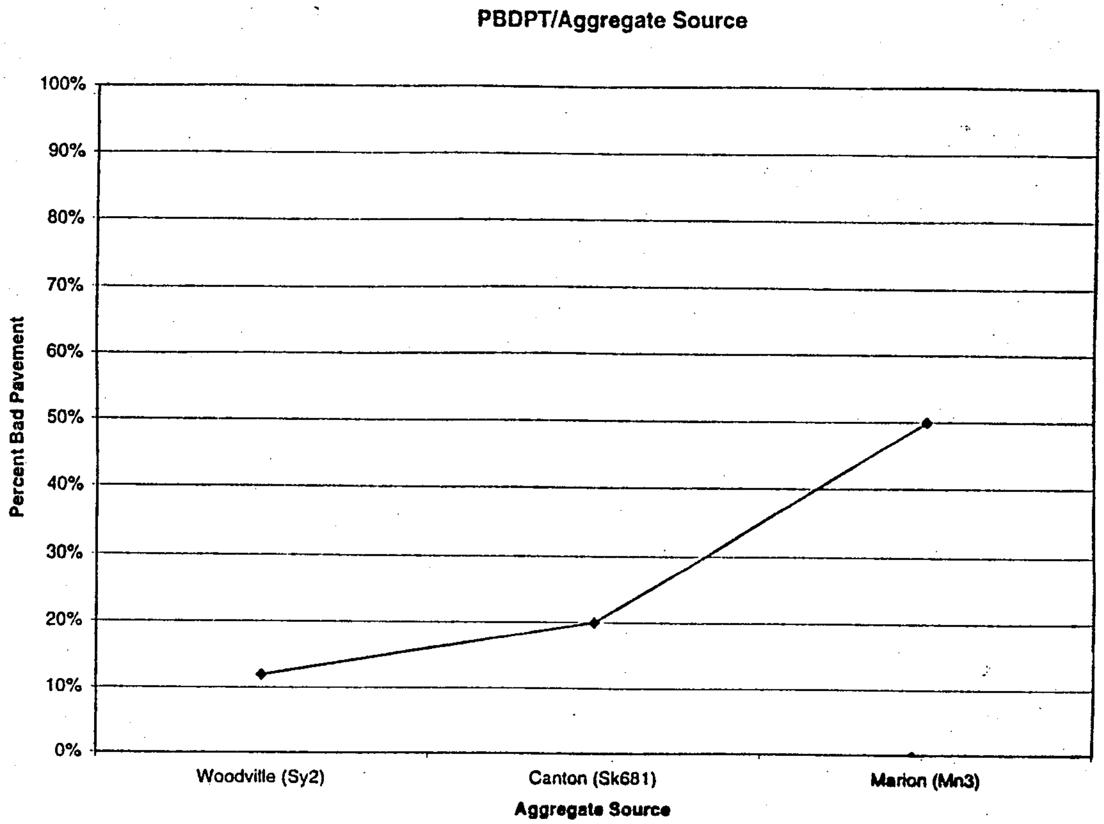
A8 Probability of Validity for Percent of Joint Pavement that is Bad (PJPTBD).

The "**Dependent Variable,**" "**PJTPTBD**" is the variable analyzed, the amount of bad joint pavement for all sections. The "**Pr > F**" of 0.010 shows the probability of validity; since any value under .05 is accepted as valid by current statistical procedures, this overall model is considered valid. The "**R-square**" of 0.401 reports the strength of the model; 40.14% of the percent bad pavement is predicted with this model. The "**PJPTBD Mean**" is the average amount of bad joint pavement of all pavement sections.

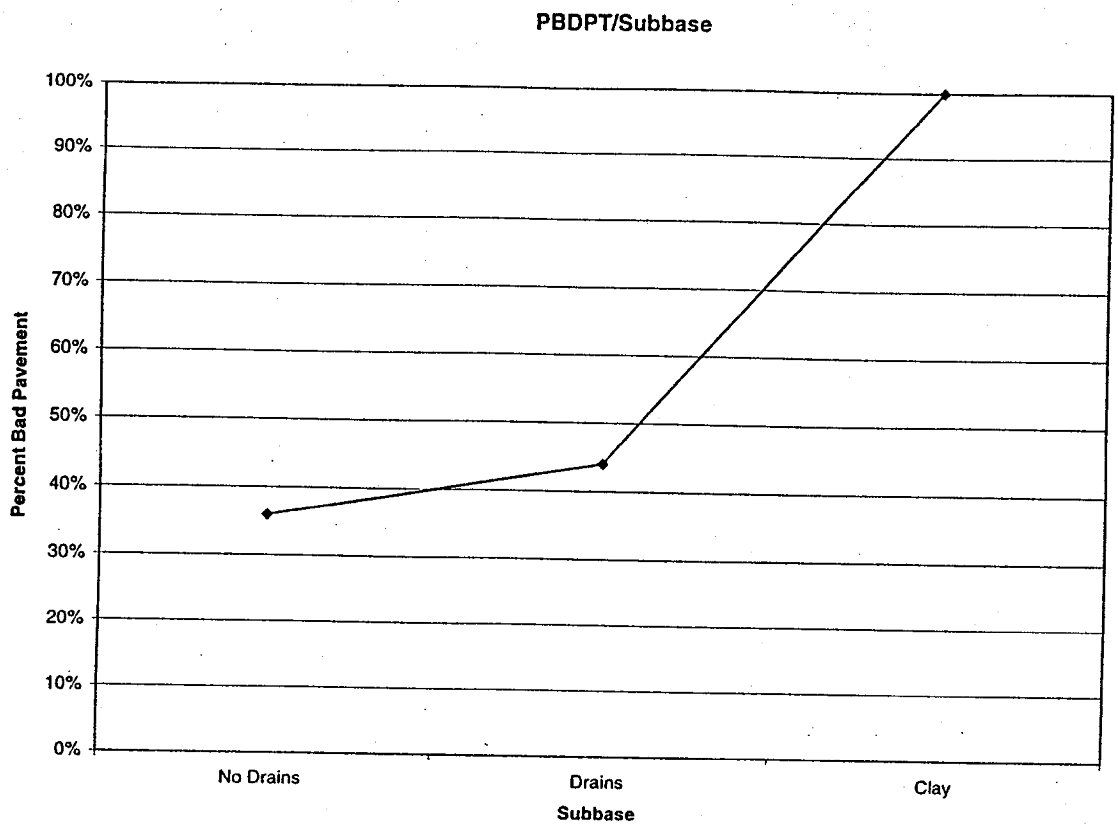
The "**Pr > F**" values for **Size** (0.0160), **Sealant** (0.0484), and **Source** (0.0166) indicate these individual variables do have a definite effect on the percent of bad pavement in a section. The "**Pr > F**" values for **Subbase** (0.1093), and **Vapor** (0.8781) indicate these variables are not significant, and do not conclusively affect the amount of bad joint pavement of all test sections.

APPENDIX B

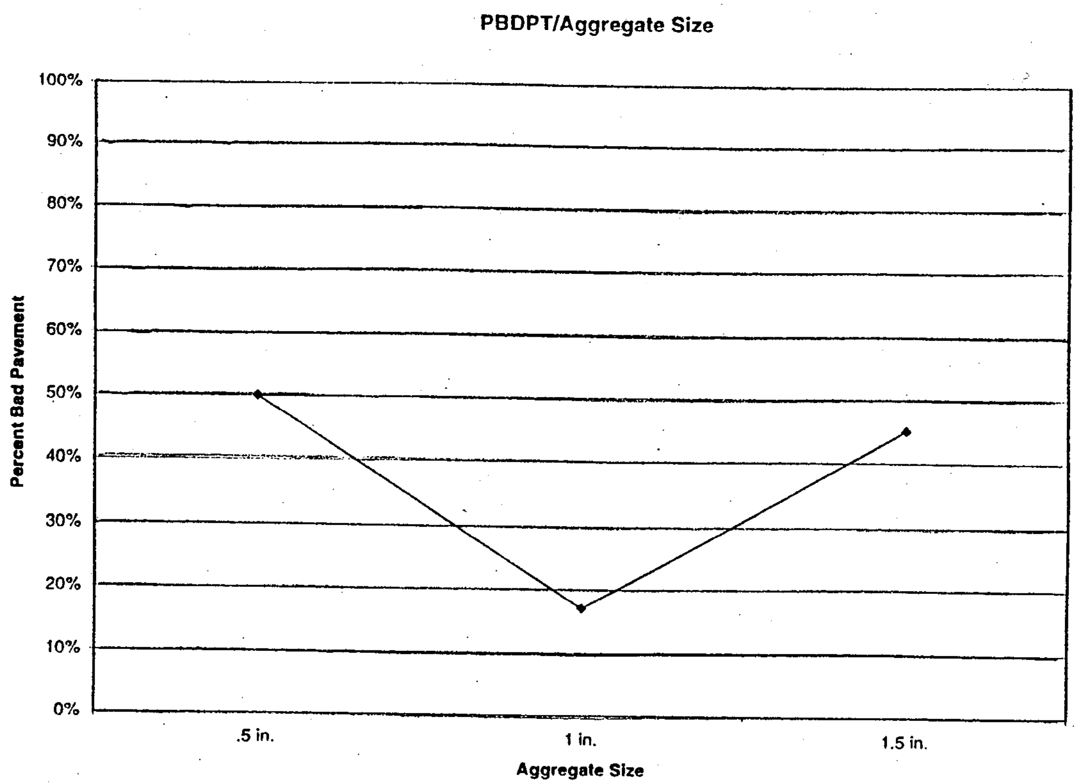
PERCENTAGES OF DEFECTIVE PAVEMENT



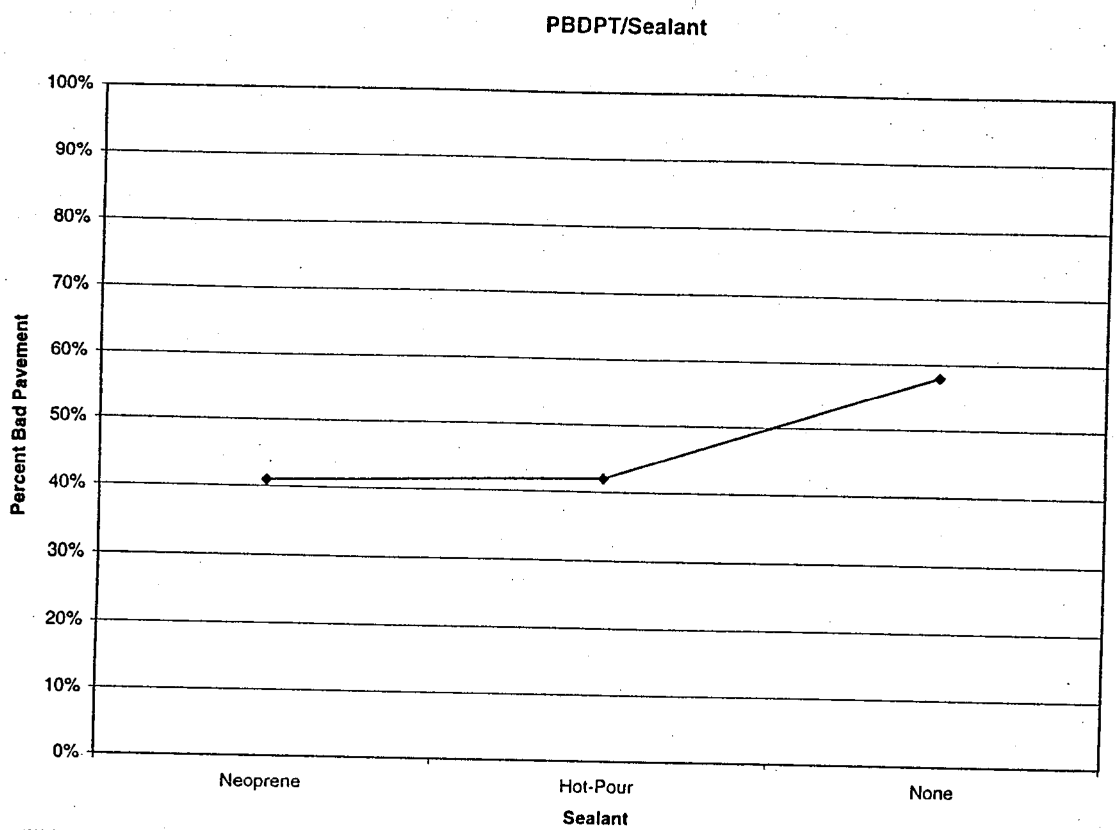
B1 PDBPT/Aggregate Source (Percent Bad Pavement by Aggregate Source).



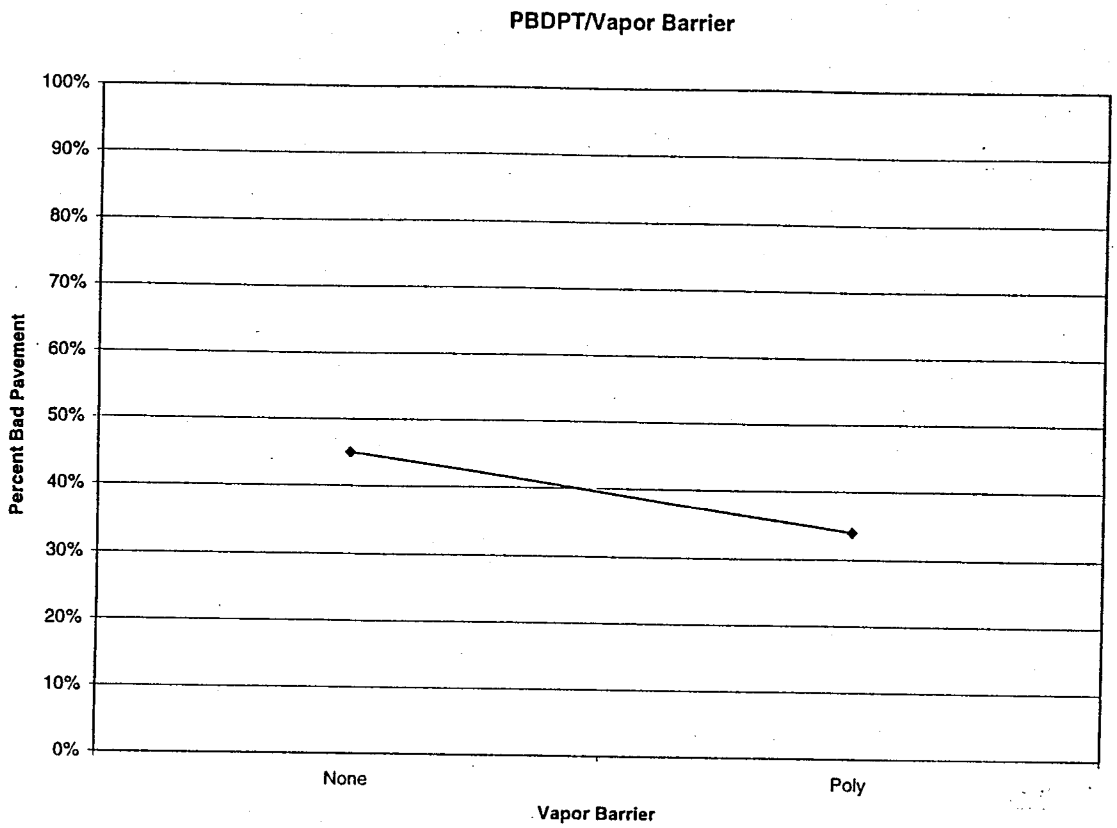
B2 PBDPT/Subbase (Percent Bad Pavement by Subbase).



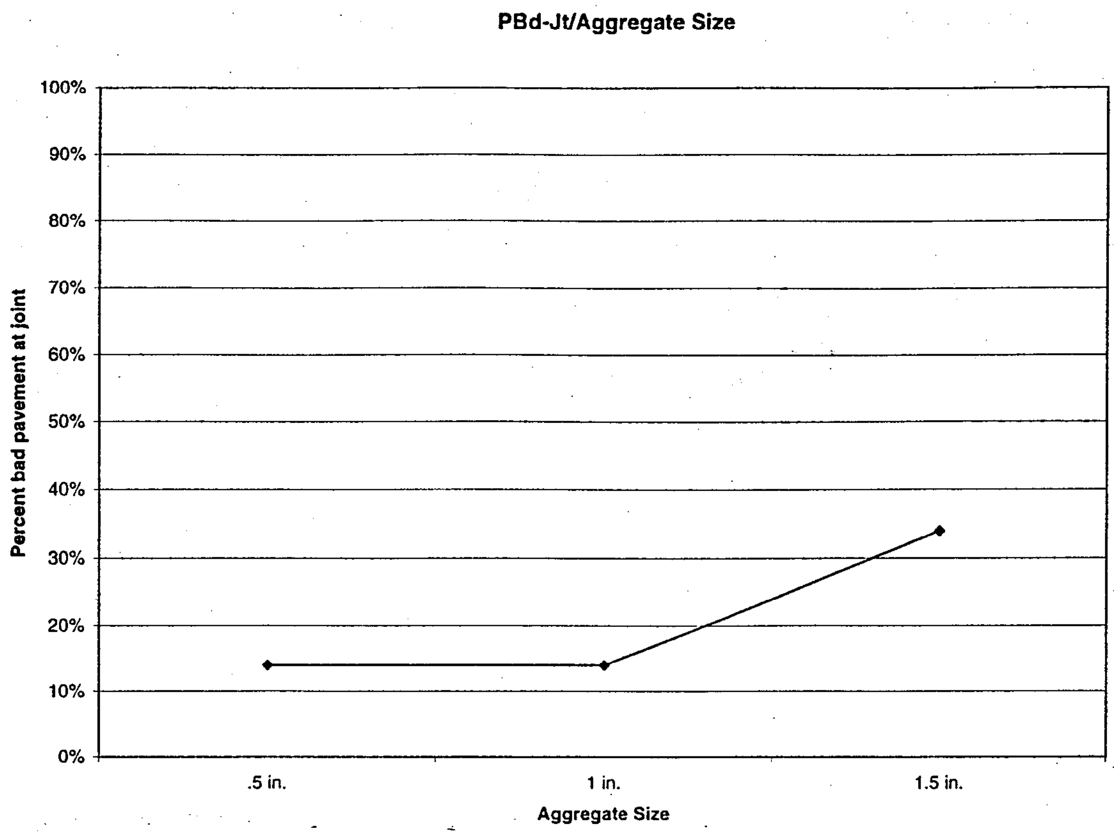
B3 PBDPT/Aggregate Size (Percent Bad Pavement by Aggregate Size).



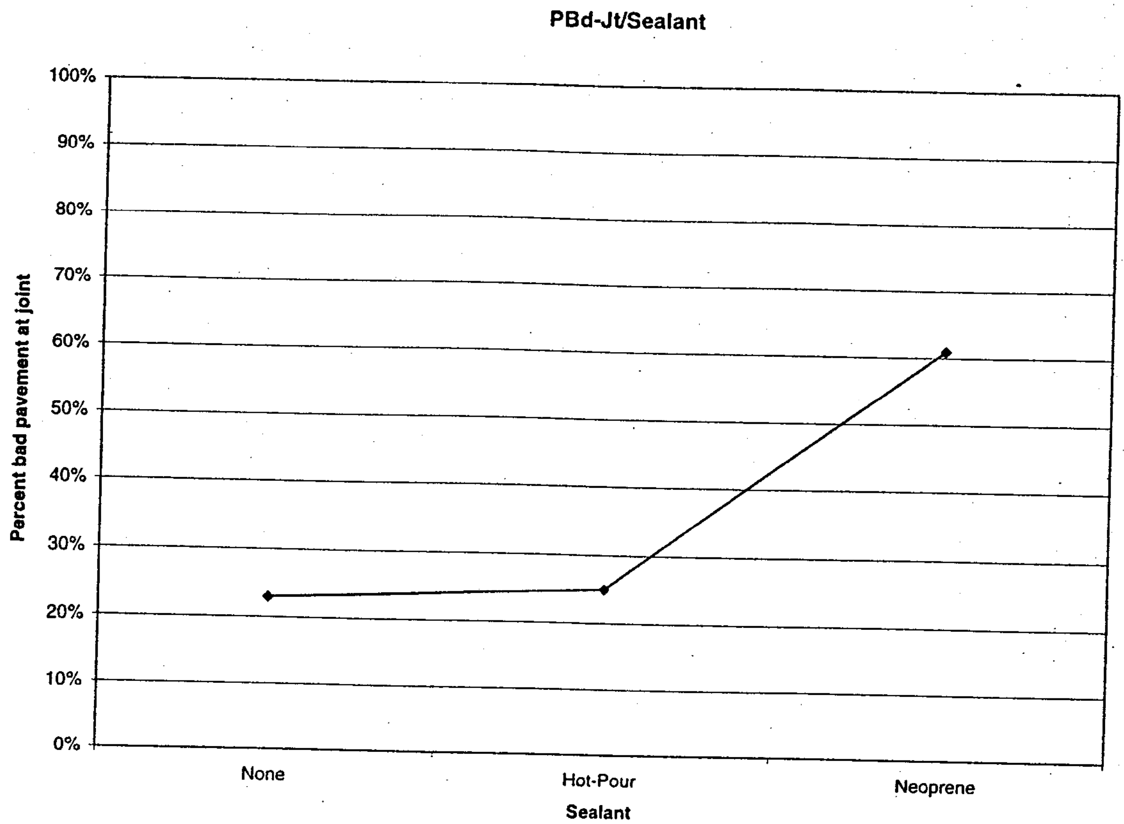
B4 PBDPT/Sealant (Percent Bad Pavement by Sealant).



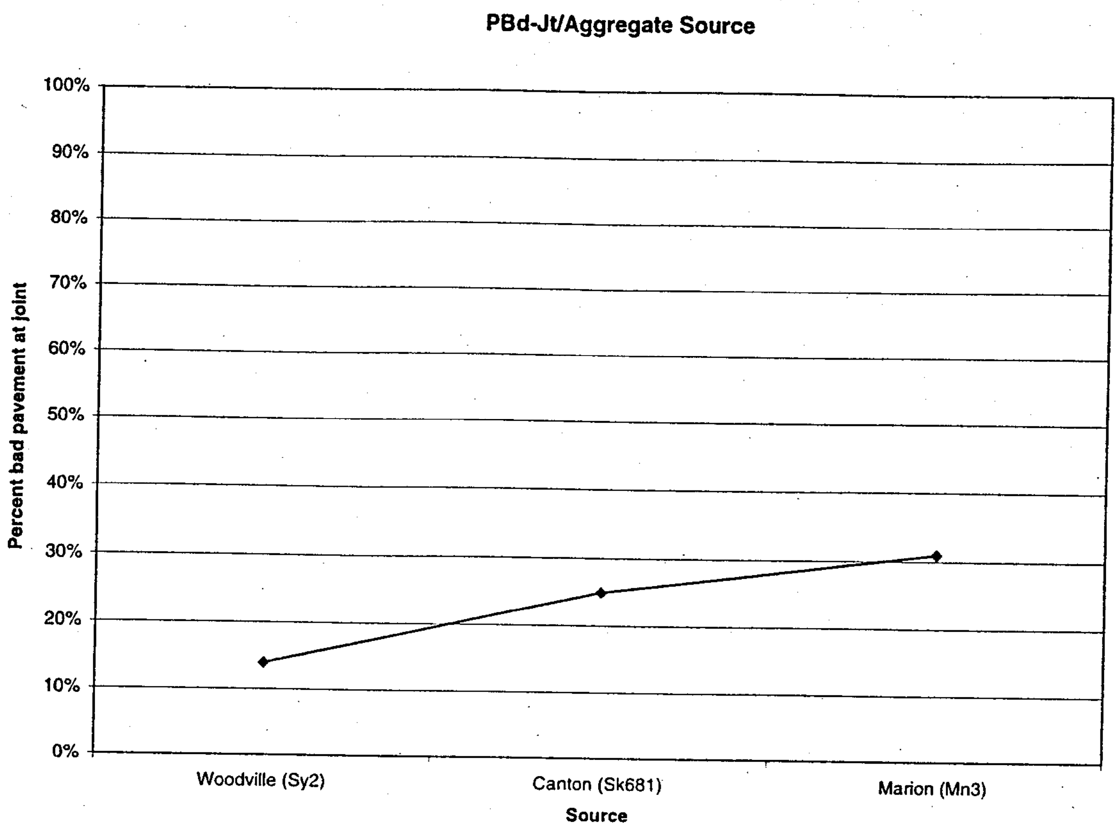
B5 PBDPT/Vapor Barrier (Percent of Bad Pavement by Vapor Barrier).



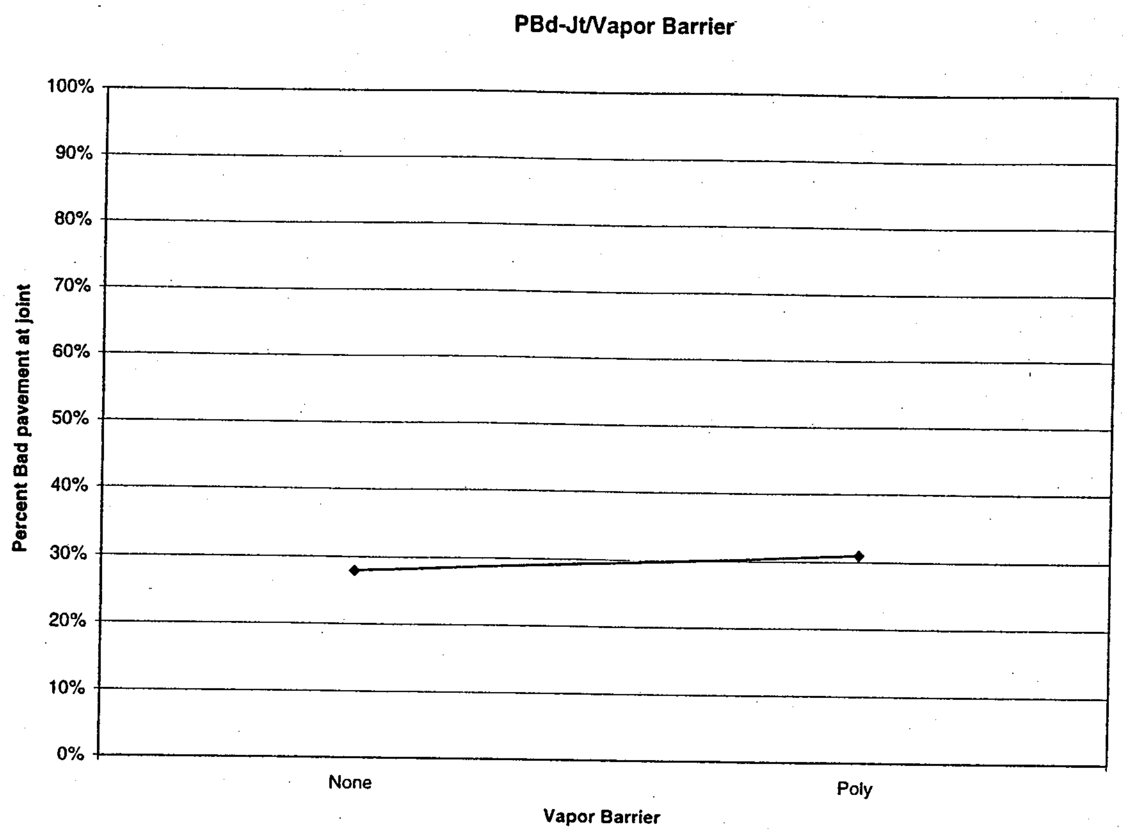
B6 PBd-Jt/Aggregate Size (Percent Bad Pavement at Joint by Aggregate Size).



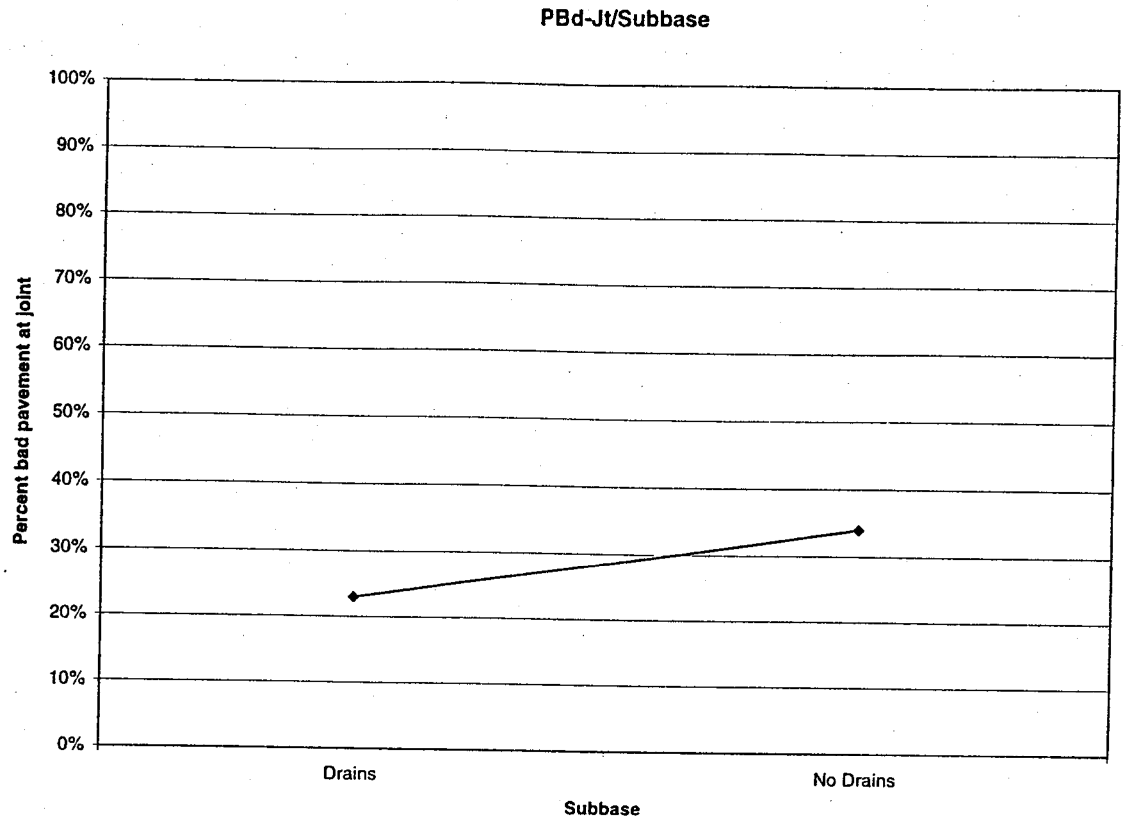
B7 PBd-Jt/Sealant (Percent Bad Pavement at Joint by Sealant).



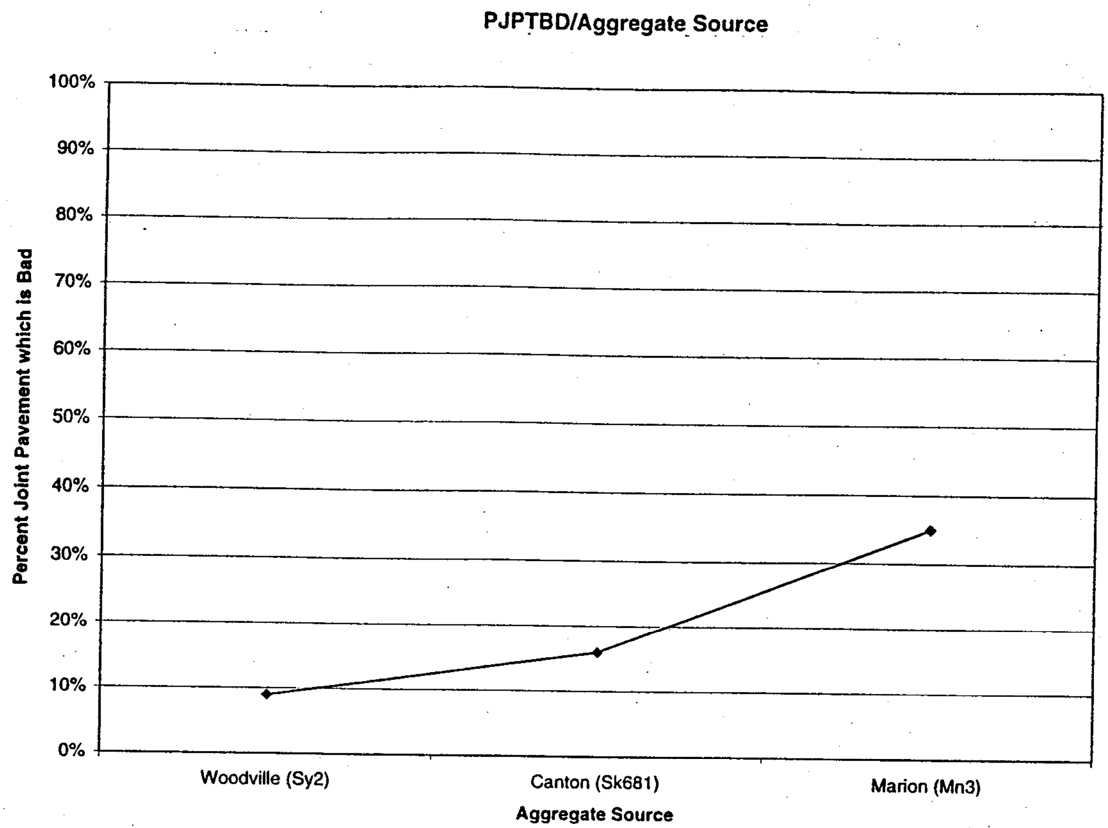
B8 PBd-Jt/Aggregate Source (Percent Bad Pavement at Joint by Aggregate Source).



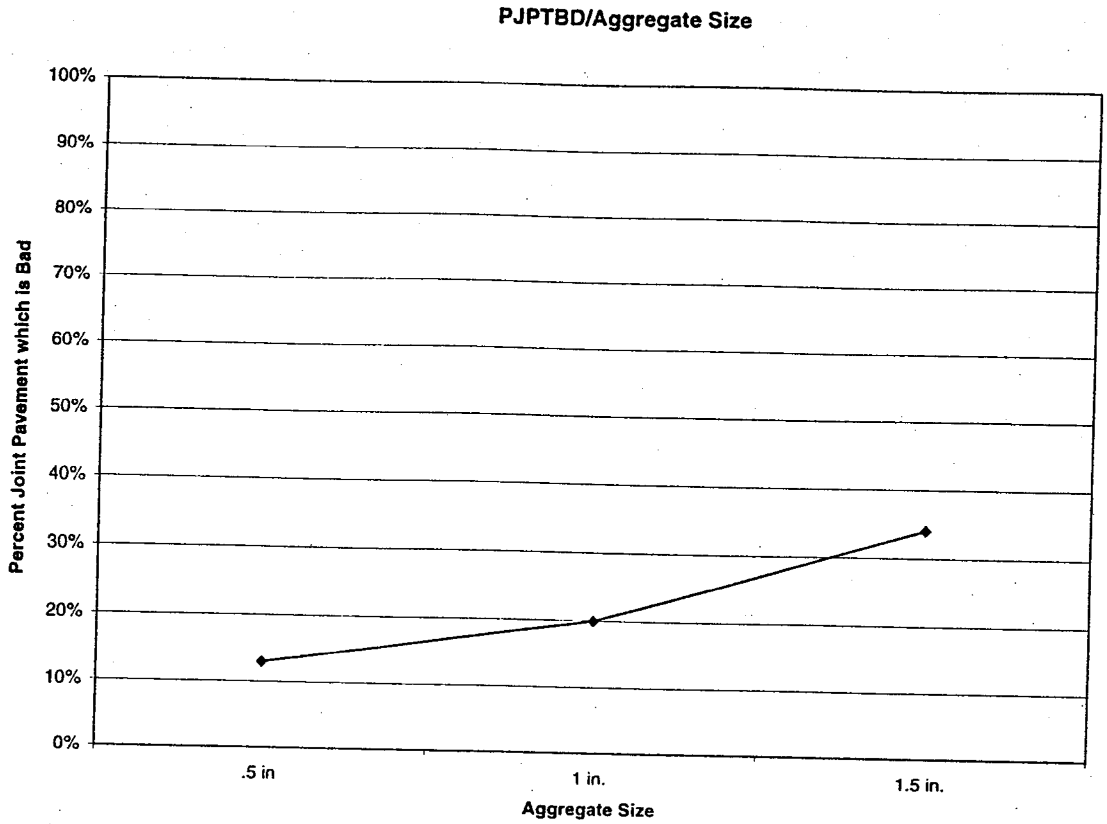
B9 PBd-Jt/Vapor Barrier (Percent Bad Pavement at Joint by Vapor Barrier)



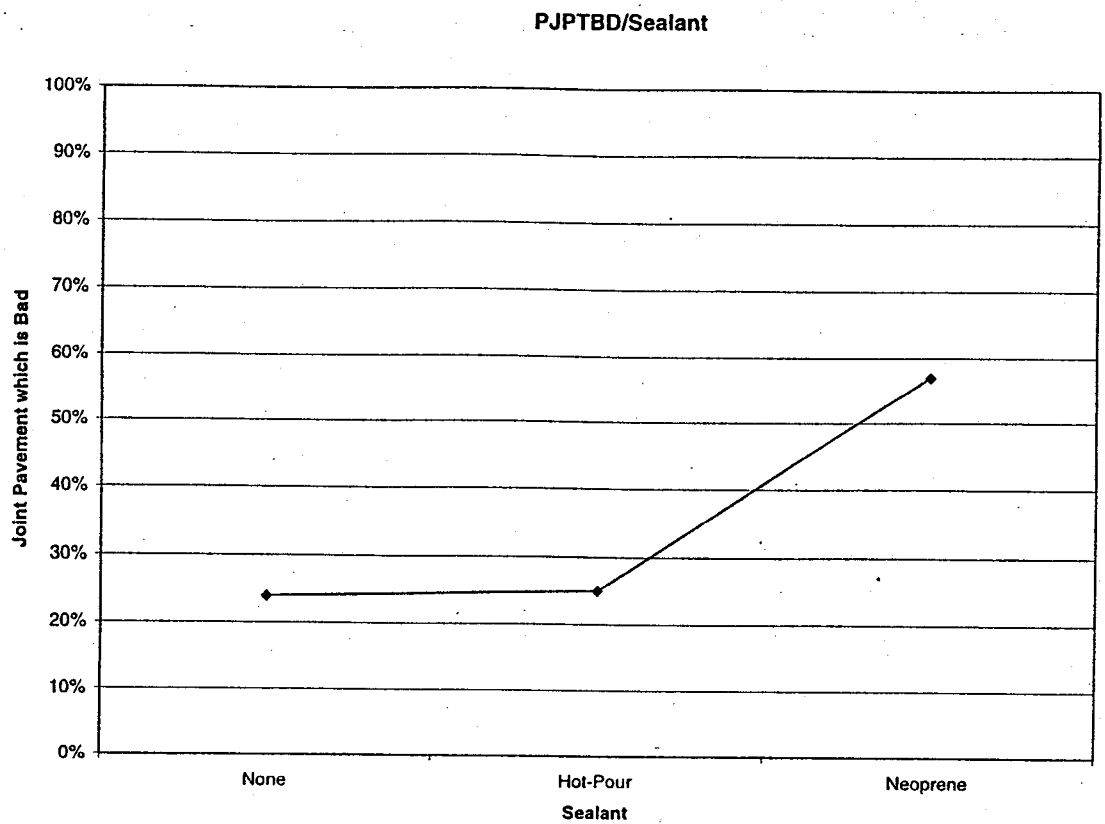
B10 PBd-Jt/Subbase (Percent Bad Pavement at Joint by Subbase).



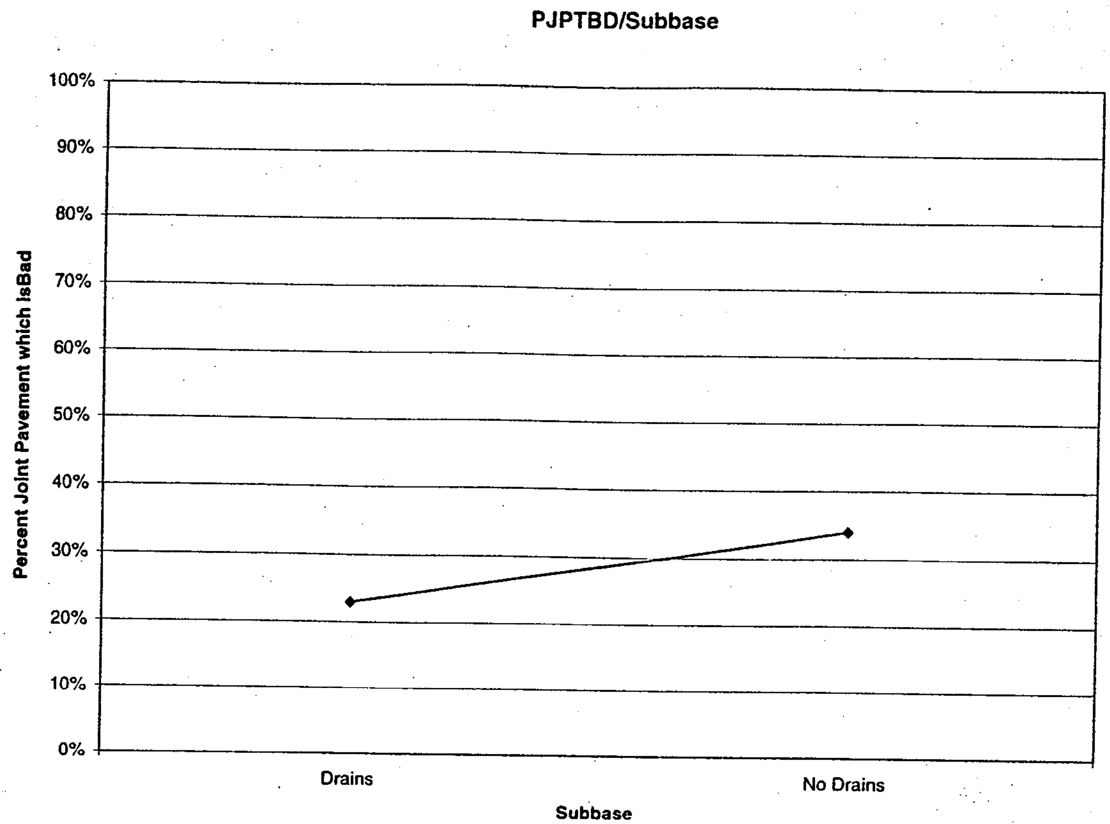
B11 PJPTBD/Aggregate Source (Percent Joint Pavement which is Bad by Aggregate Source).



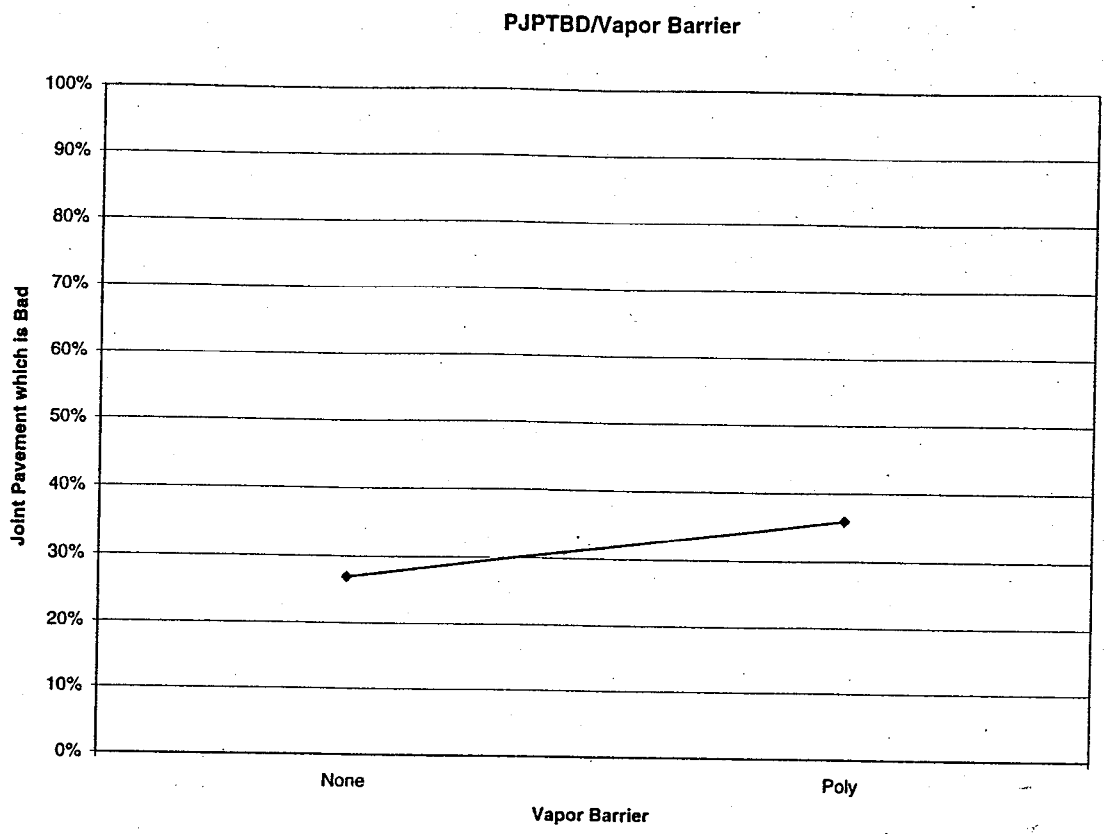
B12 PJPTBD/Aggregate Size (Percent Joint Pavement which is Bad by Aggregate Size).



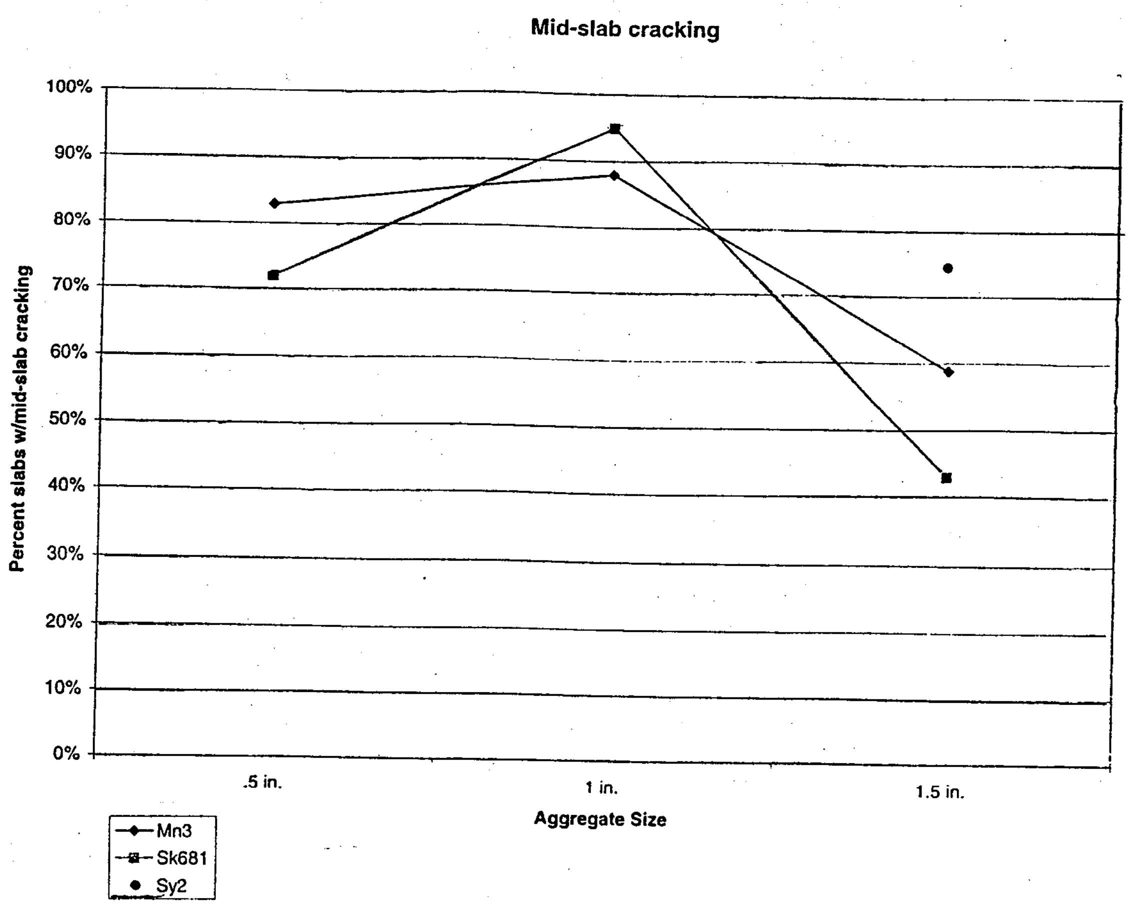
B13 PJPTBD/ Sealant (Percent Joint Pavement which is Bad by Sealant).



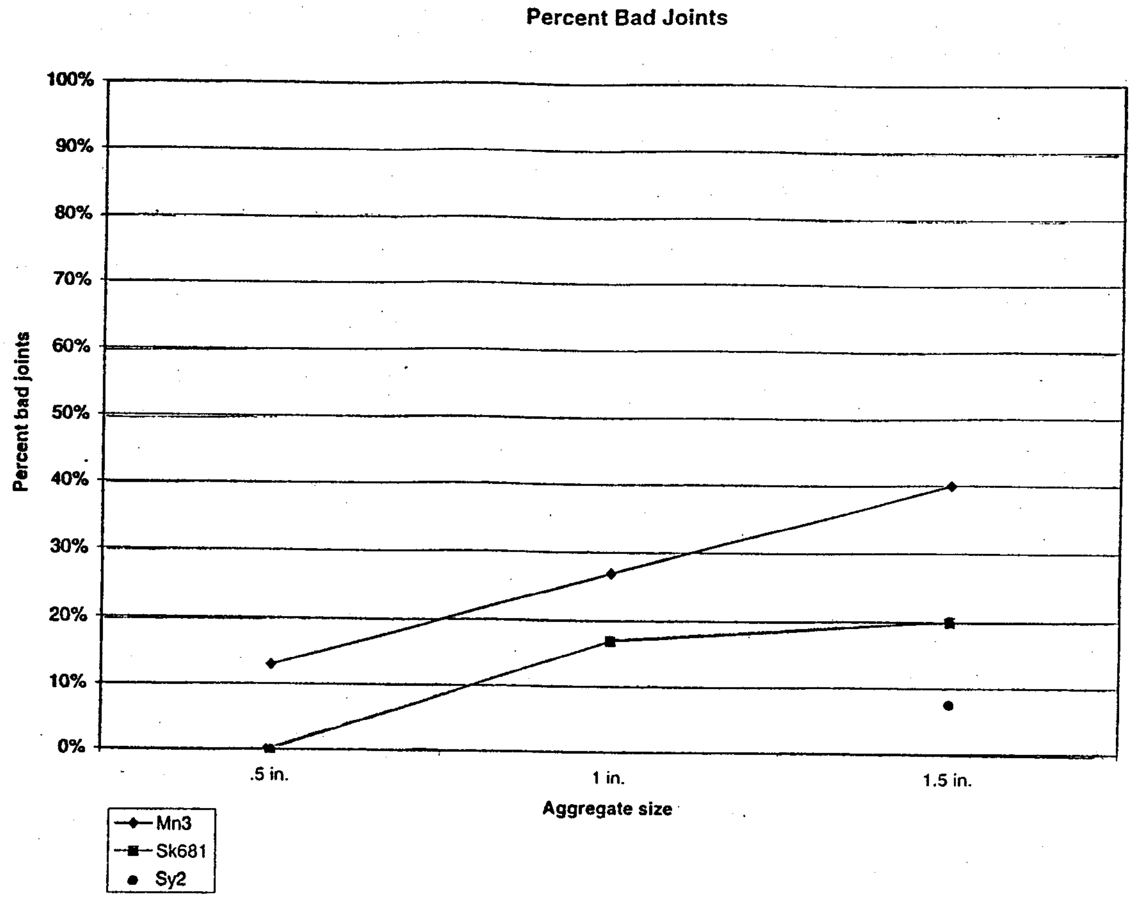
B14 PJPTBD/Subbase (Percent Joint Pavement which is Bad by Subbase).



B15 PJPTBD/Vapor Barrier (Percent Joint Pavement which is Bad by Vapor Barrier).



B16 Mid-slab Cracking (Percent of Sections with Mid-Slab Cracking).



B17 Percent Bad Joints (Percent of Joints Repaired).

APPENDIX C

AGGREGATE MANUFACTURERS' SURVEY

Bowling Green State University

Department of Technology Systems
College of Technology
Bowling Green, Ohio 43403-0302-01
(419) 372-2439

May 22, 1998

Gary Radabaugh
Rupp Construction
18228 Fulton Road
Marshallville, OH 44645

Sample

Dear Aggregate Producer:

Recently, ODOT has introduced a revision in the coarse aggregate requirements for all projects which have more than 10,000 square yards of Portland Cement concrete pavement. Proposal Note 451-97 requires that only #57's be used in these larger projects and that the aggregate passes the ASTM 666 freeze-thaw test which measures for D-Cracking susceptibility. ODOT has a concern as to the impact of this proposal change.

As part of a larger study on D-Cracking, we have devised the attached questionnaire addressing various issues that are potentially impacted by this new proposal note. Please take a few moments to complete the attached questionnaire and return it to us in the enclosed stamped envelope.

Sincerely yours,

Travis Chapin

L. Travis Chapin, P.E.
Principal Investigator

Attachments: Proposal Note 451-97
Survey Questionnaire

P.S. If you are curious about the larger study on D-Cracking, this is what is happening. In 1975, an experimental pavement was placed on Route 2 in Vermilion, Ohio. The main focus of the study was to introduce numerous variables (aggregate source and size, cements, drainage, joint treatments, etc.) and measure their impact on D-Cracking deterioration. Bowling Green State University has an ODOT contract to do this study. This preliminary study has been expanded to cover other related issues in the D-Cracking investigation. This questionnaire addresses one of these issues. In addition, we are looking at the impact on the initial construction costs and the eventual rehabilitation costs resulting from the use of large size, excellent quality coarse aggregates. If for any reason you would like to discuss this further, please give me a call at (419) 372-2837.

703.13 Coarse Aggregate for 451, 452 and 453 Portland Cement Concrete Pavement.

In addition to the requirements of 703.02, the following aggregate requirements shall apply for 451, 452 and 453.

If crushed air cooled blast furnace slag is selected for the concrete coarse aggregate, No. 57 or 67 size shall be used.

Where gravel or limestone is selected and the total combined quantity of the listed items is greater than 8000m2 (10,000 square yards), the coarse aggregate shall be Size No. 57 or 67. If the total combined quantity of the listed items is less than 8000m2 (10,000 square yards), the coarse aggregate shall be one of the following sizes: No. 7, No. 78, No. 8, No. 57 or No.67. Where Size No. 7, No. 78 or No. 8 is selected, the coarse aggregate shall be tested in accordance with 703.02. If gravel or limestone Size No. 57 or 67 is selected in either of the above cases, the coarse aggregate incorporated into the concrete shall meet 703.02 and be tested in accordance with ASTM C 666 Procedure B. The area generated under the curve obtained by plotting the expansions of test specimens verses the number of test cycles shall not exceed 2.05 at 350 or less cycles.

The validity of results of freeze thaw-resistance testing will be as outlined below:

Range of Area under Curve (1)	Status of Source Approval
0.00 - 1.00	Valid for two years from date approved (2)
1.01 - 2.05	Valid for one year from date approved (2)
2.06 - 4.00	Not Approved, one retest allowed (3)
> 4.00	Not Approved, no retesting allowed (3)

Notes :
(1) As measured at 350 Cycles.

(2) If a notable change in the properties of aggregate originating from the affected source is determined from quality control testing, a retest of freeze-thaw resistance may be requested before the original expiration date. The determination to require retesting will be made by the Office of Materials Management.

(3) Except as noted, retesting will not be done unless the producer of the material sends a written request to the Department with substantiation that significant changes in operation have been made (e.g., new processing equipment, material from a new ledge, etc.).

The Office of Materials Management will maintain a list of approved sources. Copies of the listing will be sent to each District Engineer of Tests, the Ohio Aggregate Association and the Ohio Ready Mix Concrete Association on a monthly basis. Interim reports on samples undergoing testing may be obtained from the Cement and Concrete Section.

Presently, ODOT is requiring #57 coarse aggregate which conforms to the new Proposal Note 451-97 (passes ASTM 666 freeze-thaw test) on all projects which have over 10,000 square yards of concrete pavement. In the aggregate marketplace, there are concerns over the effect that this may have on the market. To help ODOT address these concerns, we have developed this survey which we hope will provide the aggregate industry and ODOT a perspective on these issues. Information gathered with this survey will be reported in summary form and individual answers will be confidential unless approached by provider.

1) Are you aware of the new proposal?

__Yes, very knowledgeable
__Yes, knowledgeable
__Yes, somewhat knowledgeable
__No, have not heard of it

Comments

2) Do you think the new specifications will have an impact on your aggregate operations?

__Yes, a significant positive impact
__Yes, a slightly positive impact
__No impact at all
__Yes, a slightly negative impact
__Yes, a significant negative impact
__Don't know

Comments

3) In general, does your aggregate pass the ASTM 666 freeze-thaw durability test?

__Always passes
__Passes most of the time
__Rarely passes
__Never passes
__Our quarry has not been tested
__Don't know
__Other ____________________

Comments

4) Because #8 aggregate is smaller than #4 and #57 aggregate, it would be logical that #8 aggregate is/are more expensive than #4 and #57 aggregate to produce.

Is this true?
__Almost always true
__Sometimes true
__Almost never true

Comments

5) Because #8 aggregate is small than #4 and #57 aggregate, it would be logical that the production of #8 aggregate yields significantly more waste fines.

Has this been your experience?
__Yes, significantly more waste fines
__No, not significantly more wast fines
__No effect at all

Comments

6) The availability of #8 aggregate vs. #4 aggregate vs. #57 aggregate is almost always a factor of market demand.

Comments

__Almost Always
__Sometimes
__Almost never

7) In an average year (over the past three years), what has been the cash price at your plant picked up by the customer for:

Comments

#4 $______/Ton
#57 $______/Ton
#6 $______/Ton
#8 $______/Ton

8) If market demand was not a factor, which aggregate gradation would you prefer to produce? (Check one)

__#4 __#57 __#6 __#8

Comments

__Other (Explain)
__Makes no difference

9) Does it make any difference from a production standpoint (with no consideration of market demand) whether you produce #4, #57, #6, or #8 aggregate?

__No
__Yes

Comments

Please briefly explain

10) Based on 100 percent, what is the ratio of #4, #57, #6, and #8 coarse aggregate that your plant can produce at maximum capacity?

Example: ____#4 ____#57 ____#6 ____#8 ____ Other (Specify) ________________

____#4 ____#57 ____#6 ____#8 ____ Other (Specify) ________________

__Makes no difference, there is no optimum

11) What percentage of your aggregate is sold to ODOT projects?

___

Company ____________________ Quarry Location ____________________

Contact Name ____________________

Please feel free to include any additional comments on a separate sheet of paper.

L. Travis Chapin, Bowling Green State University (419) 372-2837