specific pieces of equipment, average energy consumption figures were used. Values published by the Asphalt Institute were the primary source.

The construction operations considered were materials removal, production, and transportation. It was assumed that, once the material reached the plant site, the mixing and placing operations balanced each other and only production of the component materials, such as crushing and screening, was significant.

The contractor would have used the same methods to break and remove the D-cracked pavement if it was to have been salvaged or disposed of. There was a net requirement of 130 additional equivalent gallons of gasoline for the concrete recycling option. This was the energy consumed in removal of bituminous pavement.

Overall, materials production for recycled concrete would require more energy than materials production for conventional concrete. This amounts to about 24 500 gal of gasoline.

To complete the evaluation of energy consumed, it was necessary to determine the energy requirements for transporting the construction materials. This determination assumed a constant source whenever possible, and the source was the actual supplier for the project.

The energy requirement for transportation of construction materials was less for the recycling option than for conventional paving. This amounts to a savings of 65 300 gal of gasoline.

A summary of the energy requirements in equivalent gallons of gasoline for recycling versus conventional paving is given in the table. There was an indicated savings of 40 610 gal of gasoline of an estimated 915 860 gal for conventional concrete paving. This would operate 51 automobiles, averaging 15 miles/gal, and traveling the normal 12 000 miles/year for a full year. This, in itself, is not an overwhelming savings, but it should be noted that conventional paving would require disposal of the broken D-cracked pavement. It is possible that the material may have been disposed of at the borrow pit where the contractor's plant was located. If another disposal site were required, additional truck haul would be necessary. The energy requirement would have increased further for conventional construction.

SUMMARY

In summary, the project selection process indicated the desirability of entering into a concrete recycling project on the basis of economic and engineering factors. Experimental work with trial mixes provided the concrete design most desirable for field application, and field application resulted in a project with a surface that has good riding quality. The US-59 recycling project is considered to be a success. The scarcity of quality aggregates in this area of Minnesota is partly responsible for that. In other areas, cost and energy savings may not be realized.

ACKNOWLEDGMENT

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Relation Between Pavement D-Cracking and Coarse-Aggregate Pore Structure

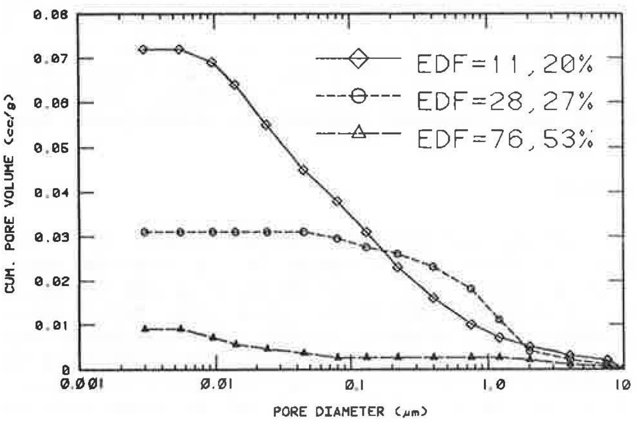
D.N. WINSLOW, M.K. LINDGREN, AND W.L. DOLCH

Previous research has developed a relation between the pore structure of a coarse aggregate and the freeze-thaw durability, measured in the laboratory, of concrete made with that aggregate. This has permitted the calculation of an expected durability factor from a knowledge of an aggregate's median pore diameter and total pore volume. This work has been extended to a consideration of in-service pavements, some of which showed D-cracking distress and some of which appeared to be durable. Forty-seven pavement sections were cored, and coarse-aggregate samples were removed and separated lithologically. Their pore-size distributions were determined by mercury intrusion. Criteria involving the expected durability factor and relative amounts of good and bad aggregate fractions in the pavement were correlated with the extent of observed D-cracking. These correlations were distinctly superior to absorption measurements in identifying bad aggregates. It is suggested that the established criteria might be used to predict the performance to be expected from aggregate sources and might be a valuable acceptance standard.

D-cracking is an important durability problem in portland cement concrete (PCC) that is exposed to freezing conditions. It is particularly prevalent in pavements. It occurs when water in the coarse-aggregate pores freezes and causes sufficient expansion to crack the concrete. When D-cracking occurs in a pavement slab, it continues until the entire slab is destroyed. There exists no practical way to stop its progress. The only measure effective against it is the exclusion from the concrete mixture of those aggregates susceptible to the problem. Many agencies that have control over the materials used in paving perform regular tests on the coarse aggregate in an effort to screen out material that will cause D-cracking. Nevertheless, this form of distress still occurs in pavements made from materials that have passed current testing procedures. It is obvious that a more diagnostic test method is required.

In earlier work (1), the relation between the pore structure of an aggregate and its susceptibility to D-cracking was examined. The pore structures of a variety of crushed-stone coarse aggre-

Figure 1. Pore-size distributions of aggregates in a typical pavement core.



gates were determined by mercury intrusion porosimetry. The D-cracking resistance of the aggregates was determined by freezing and thawing concrete made with them according to a standard testing procedure (ASTM C-666A). Certain aspects of an aggregate's pore structure were found to correlate with the aggregate's freeze-thaw durability, and the following predictive equation was developed:

$$EDF = 0.579/PV + 6.12 \times MD + 3.04 \qquad (1)$$

where

EDF = expected durability factor of aggregate (a larger value implies greater durability),
PV = aggregate's total intrudable pore volume of pores larger than 4.5 nm in diameter (cm^3/g), and
MD = median diameter of the intrudable pores larger than 4.5 nm in diameter (μm).

This relation indicates that the resistance to D-cracking is degraded by either a sufficiently large pore volume or a sufficiently small average pore size or a combination of these.

This earlier work was based on freeze-thaw performance observed in the laboratory. However, there was some indication that a knowledge of the pore structure of an aggregate could be used to predict the field performance of a pavement made with that aggregate. Furthermore, it appeared that measuring the pore-size distribution of an aggregate might be a superior diagnostic test for identifying nondurable aggregates. The work reported here was carried out to illuminate these important points.

EXPERIMENTAL PROCEDURES

The general plan of the research was to survey existing PCC highways, to note their condition with respect to D-cracking, and to remove cores from them. The pore-size distributions of the aggregates in the cores were then determined, and the predictive formula was used to find the EDF. Finally, a relation between the observed pavement condition and the measured EDF was established.

This study was restricted to four-lane highways in Indiana that had not been overlaid with a bituminous mixture. The former restriction was made to ease traffic problems while coring, and the latter one was made so that the surface could be observed for indications of D-cracking. Forty-seven pavement sections were surveyed and sampled. Each section was constructed under a separate contract and was typically 10-20 miles in length. All had a maximum coarse-aggregate size of either 1.5 or 2.5 in, and the coarse aggregate was either crushed stone or gravel. The distribution of pavement ages at the time of sampling is given below:

Age (years)	No. of Sections Sampled
0-5	1
6-10	5
11-15	19
16-20	12
21-25	7
26-30	3

The cores from each section were sawed in half, axially, and one half was crushed. Pieces of the coarse aggregate were removed from the crushed material, adhering mortar was removed when necessary, and the aggregate was washed and oven dried. It was then separated into visually distinct lithologic categories for each core. A typical core containing crushed stone had one to three different types of aggregate present, and a typical core containing gravel had four to eight different types. The sawed face of the uncrushed half of each core was polished and examined to give an approximate percentage of each type of coarse aggregate present in each core.

The pore-size distribution of each type of aggregate removed from the samples was determined by mercury intrusion porosimetry. The average of the contact angles previously reported (1), 124°, was used to calculate the pore diameters. Several pieces of the same aggregate type were tested simultaneously, as described previously (1). The EDF of each type of aggregate in each pavement section was calculated from its pore-size distribution by using the equation presented earlier.

The pore-size distributions of the aggregates from a typical core are shown in Figure 1. A total of 314 such distributions were determined during this research and are given elsewhere (2). A summary of the percentages of each type of aggregate in each core along with their EDFs is also too extensive to present here and may be found in the same report.

ANALYSIS OF DATA

One combined EDF, representative of each pavement segment, had to be obtained from the various EDFs found in that segment. A simple average, or a weighted average, of all EDFs was found to be unsatisfactory for this purpose. This was so because a few large EDF values could overwhelm the average whereas there were still a sufficient number of low-EDF aggregates present to cause distress. It was found that, when a sufficient amount of poor aggregate (low EDFs) was present, the average of only these poor rocks should be used to represent the segment. Conversely, when enough good material (high EDFs) was present, the average of only the good rocks was the appropriate value to represent the pavement.

Thus, the problem of finding a single EDF value that was representative of the aggregate in a pavement had two important aspects. One was to determine the maximum EDF value that a nondurable rock type could have. The second was to determine how much of a poor rock type must be present to cause poor pavement performance. These two questions are not independent, since the amount of poor rock in a segment depends on the definition of poor--i.e., the EDF that divides good from poor.

These two questions were answered simultaneously by a trial-and-error method. First, an EDF that divided good and poor aggregates was chosen. Next, the analysis of each core was used to determine the amount of good and poor aggregate present in each pavement section. Then a maximum limit on the amount of poor aggregate that could be present, without causing distress, was selected.

If a pavement section contained more than this maximum limit of poor aggregate, the average EDF of the poor portion of the aggregate in that section was used to represent the entire pavement section. If a section had less than this limit, the average EDF of the good portion of its aggregate was taken to represent the section.

If the EDF that divides good and poor aggregate had been selected properly, and if the maximum tolerable limit on poor aggregate had also been selected properly, then the resulting representative EDF for the section should have correctly described the actual condition of the pavement. That is, all pavements with a representative EDF below the good-poor dividing line should show D-cracking distress, and all pavements with a representative EDF above the dividing line should be free of D-cracking. If pavement sections with a poor representative EDF were observed to be in good condition, or vice versa, this meant that either the selection of the dividing EDF or the tolerable percentage of poor aggregate, or both, were improper.

The above technique used arbitrarily selected good-poor dividing lines and tolerable limits for poor aggregate plus laboratory measurements on the aggregates to "predict" the condition of each pavement section. This "prediction" was then compared with the observed pavement condition. The selection of both the good-poor division and the tolerable amount of poor aggregate was adjusted until the prediction most closely corresponded to the observed condition for all 47 pavement sections.

RESULTS

The best combination of a good-poor dividing line and maximum acceptable amount of poor aggregate was found to be that in which there was an EDF of 50 and no more than 10 percent of the aggregate had an EDF below this value. In other words, 90 percent of the aggregate should have an EDF greater than 50 if D-cracking is to be avoided. These criteria were found to be equally applicable to crushed stone and to gravel coarse aggregates. They have been shown to apply only during the first 30 years of pavement life. The following table compares observed performance (summer 1979, when the pavements were cored) and predicted performance by using these criteria:

No. of Pavement Sections	Observed Performance	Predicted Performance
29	D-cracked	Nondurable
11	Not D-cracked	Durable
7	Not D-cracked	Nondurable

DISCUSSION OF RESULTS

In this study, no attempt was made to differentiate between aggregates on the basis of how rapidly they will produce a D-cracked pavement. The criteria that were developed are of the "yes-no" variety. They differentiate between aggregates that will produce D-cracking within 30 years and those that will not. The criteria do not allow one to say that a certain aggregate will produce D-cracking within 10 years while another aggregate will require 20 years.

There were not a sufficient number of surveyable pavement sections in each age group to permit the further refinement of a prediction of the time required to display D-cracking. Furthermore, the rate of deterioration caused by a nondurable aggregate is likely to be associated with the environmental conditions surrounding a given slab and not only with the pore structure of the aggregate within the slab, and the criteria that have been presented concern themselves only with an aggregate's pore structure.

There is also an economic consideration that may be rapidly making a prediction of the rate of deterioration an academic exercise. The combination of climbing maintenance costs and increasingly scarce maintenance funds may presage the time when resurfacing a pavement solely because of D-cracking cannot be economically tolerated. Thus, it may not be important whether a pavement deteriorates in 10 or 20 years, since either possibility may be unacceptable.

This study only considered cores with a maximum aggregate size of either 1.5 or 2.5 in. No others were available. Approximately 75 percent of the cores contained 2.5-in maximum sizes. It is not felt that there were a sufficient number of cores with each maximum size to attempt the development of a separate criterion for each. Furthermore, most of the aggregates that were examined would probably not benefit from a reduction in size. In any event, the criteria presented appear to be equally applicable to both maximum sizes.

The results reported here are for either crushed limestone or gravel aggregates. These aggregates had pore sizes that were relatively tightly grouped about their median pore diameter. This median diameter varied over several orders of magnitude for the different aggregates examined. However, whatever the median might be, the pore diameter that separated the largest 25 percent of the pores from the rest was almost always less than five times as large as the median pore diameter. To put it another way, the majority of the pore volume in any aggregate usually resided in pores whose sizes were grouped within an order of magnitude of the median diameter.

Some aggregates, particularly slags, have a much wider spread in pore diameters. Only a few cores containing aggregate with widely spread pore sizes were examined. In these few, it appeared that this spread was advantageous and that it caused the aggregate to be more resistant to D-cracking than its EDF would predict. However, not enough pavements with such aggregates have been investigated quantitatively to include this parameter of their pore structure in the EDF calculation. Therefore, the findings reported here should be restricted, conservatively, to those aggregates for which the 25th percentile pore diameter is less than five times the median, or 50th percentile, pore diameter.

The preceding text table shows the result of applying the 90 percent > EDF = 50 criteria to the 47 pavement sections included in this study. The criteria always predict that an aggregate that actually shows distress is susceptible to distress. Had these criteria been used as a test to screen out susceptible aggregates, these sections would not have contained aggregate that causes D-cracking.

The sections that did not show distress at the time of sampling fall into two categories. The criteria predict that 11 sections should, indeed, be durable with respect to D-cracking. However, the criteria also predict that 7 sections should D-crack, but these 7 sections were not observed to be in distress at the time they were cored. It may be that these sections have had insufficient time to show distress, or the criteria may be conservative in that they reject all the bad aggregate and also some durable aggregate as well. Continued observation of these sections is required to answer this question.

All the aggregate in all 47 sections passed the durability acceptance test in force at the time of construction: a 24-h absorption test. Yet almost two-thirds of the sections are in distress; 3 are less than 10 years old. The criteria developed in this study consider the median pore size as well as the pore volume of an aggregate rather than merely a measure of the pore volume. Furthermore, they consider that a comparatively small amount of poor material can destroy a pavement even when a large amount of good aggregate is present. For these reasons, they are considered to do a better job of discriminating among aggregates.

If the criteria developed here were used for acceptance testing of aggregate in relation to freeze-thaw durability, it would be necessary to measure a great many pore-size distributions. Even with the advent of automatic instruments to perform the tests, this would be an expensive undertaking. However, the cost of resurfacing D-cracked pavement is vastly more expensive. Compared with this extraordinary maintenance cost, the expense of the more refined testing described here would be small.

CONCLUSIONS

1. The pore structure of the coarse aggregate in a pavement can be used to calculate an EDF that is representative of the pavement.
2. The representative EDF correlates well with the presence or absence of observed pavement D-cracking distress.
3. The pore structure of a potential coarse-aggregate source may be a useful predictor of potential D-cracking problems in pavements made with that aggregate.

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Ohio Aggregate and Concrete Testing to Determine D-Cracking Susceptibility

JOHN T. PAXTON

Several laboratory test methods were analyzed to determine their capability of indicating the D-cracking susceptibility of coarse aggregates. Two methods were modified versions of ASTM C666 A and B, two were unconfined freeze-thaw tests of the aggregate, and the remaining two were standard sodium and magnesium soundness tests. The major modification of the ASTM C666 test methods was to determine the elongation of the test specimens versus routine weight-loss determinations and/or sonic modulus determinations. Results are evaluated by plotting the percentage of expansion versus the number of cycles completed and calculating the area under the curve generated. Although 10 specimens are used in the testing, the 2 high and 2 low test results are removed before final analysis. The correlation of this test method with service records of various aggregates was found to be good; however, when the same coarse aggregates were tested in sodium sulfate, magnesium sulfate, or unconfined freeze and thaw, the results did not correlate well with the service records.

D-cracking in portland cement concrete (PCC) pavements has been and is a serious problem in Ohio. Early in 1969, it was decided that research was needed to investigate the causes of and determine solutions for this problem. In June 1969, the Ohio Department of Transportation (ODOT) entered into a cooperative research agreement for this purpose with the Portland Cement Association (PCA). By March 1972, PCA had published an interim report on D-cracking of concrete pavements in Ohio that indicated that the coarse aggregate in the concrete was a cause of the problem. In addition, this interim report indicated that a freeze-thaw test similar to ASTM C666 could be used to determine the D-cracking susceptibility of coarse aggregates. It was at this time that ODOT decided to conduct research to evaluate the capability of various test methods for indicating which coarse aggregates used in Ohio were likely to cause D-cracking. This paper deals with the results obtained by using these various test methods to test coarse aggregates from 16 sources in Ohio that had service records indicating D-cracking in less than 15 years or no D-cracking in 15 years or more. The aggregate sources and their service records are given in Table 1.

AGGREGATE TESTING BY FREEZE-THAW

Confined

The initial testing undertaken by ODOT was freeze-thaw testing of the coarse aggregate incorporated in a concrete. The freeze-thaw tests used were similar to ASTM C666 methods A and B; however, instead of a determination of the sonic modulus of concrete specimens undergoing freeze-thaw cycling, the expansions of the specimens were checked and recorded throughout the test period. It was determined from the PCA research that the specimens that exhibited the larger expansions were those made with coarse aggre-