NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
SYNTHESIS OF HIGHWAY PRACTICE 134

D-CRACKING OF CONCRETE PAVEMENTS

TRANSPORTATION RESEARCH BOARD
National Research Council

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
SYNTHESIS OF HIGHWAY PRACTICE **134**

D-CRACKING OF CONCRETE PAVEMENTS

DONALD R. SCHWARTZ
Springfield, Illinois

Topic Panel

BERNARD BROWN, *Iowa Department of Transportation*
JAMES GEHLER, *Illinois Department of Transportation*
NEIL F. HAWKS, *Transportation Research Board*
PAUL KLIEGER, *Northbrook, Illinois*
WILLIAM TRIMM, *Missouri Highway and Transportation Department*
HARVEY E. WALLACE, *Kansas Department of Transportation*
KURT DUNN, *Federal Highway Administration* (Liaison)
MICHAEL RAFALOWSKI, *Federal Highway Administration* (Liaison)

RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS IN COOPERATION WITH THE FEDERAL HIGHWAY ADMINISTRATION

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. OCTOBER 1987

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as: it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to the National Research Council is an assurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the National Research Council and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the National Research Council and its Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

NOTE: The Transportation Research Board, the National Research Council, the Federal Highway Administration, the American Association of State Highway and Transportation Officials, and the individual states participating in the National Cooperative Highway Research Program do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

NCHRP SYNTHESIS 134

Project 20-5 FY 1985 (Topic 17-08)
ISSN 0547-5570
ISBN 0-309-04412-X
Library of Congress Catalog Card No. 87-50686

Price: $7.60

Subject Areas
Pavement Design and Performance
Mineral Aggregates
Maintenance

Mode
Highway Transportation

NOTICE

The project that is the subject of this report was a part of the National Cooperative Highway Research Program conducted by the Transportation Research Board with the approval of the Governing Board of the National Research Council. Such approval reflects the Governing Board's judgment that the program concerned is of national importance and appropriate with respect to both the purposes and resources of the National Research Council.

The members of the technical committee selected to monitor this project and to review this report were chosen for recognized scholarly competence and with due consideration for the balance of disciplines appropriate to the project. The opinions and conclusions expressed or implied are those of the research agency that performed the research, and, while they have been accepted as appropriate by the technical committee, they are not necessarily those of the Transportation Research Board, the National Research Council, the American Association of State Highway and Transportation Officials, or the Federal Highway Administration of the U.S. Department of Transportation.

Each report is reviewed and accepted for publication by the technical committee according to procedures established and monitored by the Transportation Research Board Executive Committee and the Governing Board of the National Research Council.

The National Research Council was established by the National Academy of Sciences in 1916 to associate the broad community of science and technology with the Academy's purposes of furthering knowledge and of advising the Federal Government. The Council has become the principal operating agency of both the National Academy of Sciences and the National Academy of Engineering in the conduct of their services to the government, the public, and the scientific and engineering communities. It is administered jointly by both Academies and the Institute of Medicine. The National Academy of Engineering and the Institute of Medicine were established in 1964 and 1970, respectively, under the charter of the National Academy of Sciences.

The Transportation Research Board evolved in 1974 from the Highway Research Board, which was established in 1920. The TRB incorporates all former HRB activities and also performs additional functions under a broader scope involving all modes of transportation and the interactions of transportation with society.

Published reports of the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Transportation Research Board
National Research Council
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

Printed in the United States of America

PREFACE

A vast storehouse of information exists on nearly every subject of concern to highway administrators and engineers. Much of this information has resulted from both research and the successful application of solutions to the problems faced by practitioners in their daily work. Because previously there has been no systematic means for compiling such useful information and making it available to the entire highway community, the American Association of State Highway and Transportation Officials has, through the mechanism of the National Cooperative Highway Research Program, authorized the Transportation Research Board to undertake a continuing project to search out and synthesize useful knowledge from all available sources and to prepare documented reports on current practices in the subject areas of concern.

This synthesis series reports on various practices, making specific recommendations where appropriate but without the detailed directions usually found in handbooks or design manuals. Nonetheless, these documents can serve similar purposes, for each is a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems. The extent to which these reports are useful will be tempered by the user's knowledge and experience in the particular problem area.

FOREWORD

By Staff
Transportation Research Board

This synthesis will be of interest to pavement designers, materials engineers, maintenance engineers, and others concerned with design, construction, and maintenance of portland cement concrete pavements. Information is presented on the causes of and potential means for minimizing D-cracking of concrete pavements.

Administrators, engineers, and researchers are continually faced with highway problems on which much information exists, either in the form of reports or in terms of undocumented experience and practice. Unfortunately, this information often is scattered and unevaluated, and, as a consequence, in seeking solutions, full information on what has been learned about a problem frequently is not assembled. Costly research findings may go unused, valuable experience may be overlooked, and full consideration may not be given to available practices for solving or alleviating the problem. In an effort to correct this situation, a continuing NCHRP project, carried out by the Transportation Research Board as the research agency, has the objective of reporting on common highway problems and synthesizing available information. The synthesis reports from this endeavor constitute an NCHRP publication series in which various forms of relevant information are assembled into single, concise documents pertaining to specific highway problems or sets of closely related problems.

In certain areas of the country, the coarse aggregates used in concrete pavements are susceptible to disintegration from repeated freezing and thawing. This report of the Transportation Research Board describes the mechanisms of D-cracking, summarizes known materials-acceptance and design techniques that can minimize

D-cracking of new pavements, and describes rehabilitation techniques for existing pavements.

To develop this synthesis in a comprehensive manner and to ensure inclusion of significant knowledge, the Board analyzed available information assembled from numerous sources, including a large number of state highway and transportation departments. A topic panel of experts in the subject area was established to guide the researcher in organizing and evaluating the collected data, and to review the final synthesis report.

This synthesis is an immediately useful document that records practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As the processes of advancement continue, new knowledge can be expected to be added to that now at hand.

CONTENTS

ACKNOWLEDGMENTS

This synthesis was completed by the Transportation Research Board under the supervision of Robert E. Skinner, Jr., Director for Special Projects. The Principal Investigators responsible for conduct of the synthesis were Thomas L. Copas and Herbert A. Pennock, Special Projects Engineers. This synthesis was edited by Anne S. Brennan.

Special appreciation is expressed to Donald R. Schwartz, Springfield, Illinois, who was responsible for the collection of the data and the preparation of the report.

Valuable assistance in the preparation of this synthesis was provided by the Topic Panel, consisting of Bernard Brown, Testing Engineer, Iowa Department of Transportation; James Gehler, Engineer of Materials and Physical Research, Illinois Department of Transportation; Paul Klieger, Consultant, Concrete and Concrete Materials, Northbrook, Illinois; William Trimm, Division Engineer, Materials & Research, Missouri Highway and Transportation Department; Harvey E. Wallace, Engineer of Tests, Bureau of Materials and Research, Kansas Department of Transportation; and Liaison Members Kurt Dunn, Highway Engineer, and Michael Rafalowski, Highway Engineer, Federal Highway Administration.

Neil F. Hawks, Engineer of Soils, Geology and Foundations, Transportation Research Board, assisted the NCHRP Project 20-5 Staff and the Topic Panel.

Information on current practice was provided by many highway and transportation agencies. Their cooperation and assistance were most helpful.

D-CRACKING OF CONCRETE PAVEMENTS

SUMMARY

D-cracking, a form of concrete deterioration in portland cement concrete pavement, was first described in the 1930s, but within the last several years it has developed into a serious, widespread problem in certain areas of the eastern and west-central United States and in Canada.

D-cracking is associated primarily with the use of coarse aggregates in the concrete that disintegrate when they become saturated and are subjected to repeated cycles of freezing and thawing. The cracking originates in the coarse aggregate particles and then propagates through the mortar matrix surrounding the aggregate. A series of slightly inclined cracks develop in the concrete, usually starting at the bottom of the slab and working upward. Signs of D-cracking on the pavement surface include a series of closely spaced cracks that are generally parallel to transverse and longitudinal joints and cracks and to the pavement free edges and may be filled with a dark deposit at the pavement surface. Identification of D-cracking before signs are evident on the pavement surface can be accomplished only by examination of full-depth cores taken through the pavement near the intersections of longitudinal and transverse joints and cracks.

Other factors such as the fine aggregate and the brand or composition of cement used in the concrete, the various elements of pavement design, subsurface drainage, and traffic appear to have little, if any, influence on the development of D-cracking. Pavement design, subsurface drainage, and traffic, however, can influence the rate of concrete deterioration that occurs once D-cracking has developed.

There is no simple, quick, and economical test procedure for determining the D-cracking susceptibility of a coarse aggregate. The expensive and time-consuming freezing and thawing tests are the ones used by most agencies and are considered the best for this purpose. Some of the simpler, faster tests that measure aggregate property characteristics can be used as screening tests but are not adequate by themselves for positively identifying D-cracking susceptibility.

Improvements in resistance of coarse aggregates to freezing and thawing can sometimes be accomplished through aggregate beneficiation techniques, including reducing the nominal maximum size of coarse aggregate particles, mechanical separation, and blending. Reducing the maximum size of coarse aggregate particles is considered the best technique and aggregate blending is the least desirable. Any one of the three, however, can produce results ranging from completely satisfactory to completely unsatisfactory.

Preventing the development of D-cracking in portland cement concrete pavement can only be accomplished by carrying out expensive and time-consuming testing programs and by tightening materials and construction specifications to guard against the use of D-cracking susceptible coarse aggregate in the concrete. The testing for material acceptance should be done on a source-by-source or project-by-project basis.

Initially, a combined laboratory and field test program is required so that test results can be compared with field performance histories to establish meaningful and reliable failure criteria. To maximize the use of locally available coarse aggregate and minimize the cost of production, the testing should be carried out through a range of maximum sizes to determine the largest maximum size, if any, available from a particular source or for a specific project that is acceptable for use in the concrete.

There are some special considerations that can be given to concrete mixture proportioning and to pavement design that can help minimize the D-cracking problem but none that will eliminate it. Increasing the fine-aggregate content and blending durable with nondurable coarse aggregate reduces the total amount of nondurable material in the concrete. Also, the inclusion of a positive subsurface drainage system can be expected to increase the time required for D-cracking to develop and possibly reduce the severity of the resultant concrete deterioration.

Rehabilitation of existing D-cracked jointed pavement is being accomplished primarily by overlaying it with a new bituminous concrete surfacing. Adequate cleaning and repair of the D-cracked pavement before resurfacing is necessary for good performance. Both asphaltic concrete and portland cement concrete are being used in partial-depth and full-depth patching. For more heavily traveled highways, the current trend is to use portland cement concrete for full-depth patching and to reestablish load transfer. Consideration should be given to increasing the nominal overlay thickness when resurfacing a D-cracked slab to compensate for the continued loss in slab strength caused by D-cracking.

Of the three types of portland cement concrete overlays (bonded, partially bonded, unbonded) that could be used over existing D-cracked jointed pavement, there have been only two experimental projects; one is a thin bonded overlay and the other is an unbonded overlay.

Rehabilitation of existing D-cracked continuously reinforced concrete (CRC) pavement has been almost exclusively accomplished by overlaying it with bituminous concrete. In overlaying D-cracked CRC pavement, the following recommendations should be considered:

1. Rehabilitate a D-cracked CRC pavement before it requires extensive patching,
2. Maintain the continuity of the reinforcement in the CRC pavement if at all possible,
3. Remove any loose or delaminated concrete on the pavement surface and repair the area before resurfacing, and
4. For badly distressed D-cracked CRC pavements, do minimal full-depth patching, very good pavement surface cleaning, and increase the overlay thickness.

The alternative to rehabilitating a D-cracked pavement is to reconstruct it. A decision to reconstruct must be made on the basis of economics and engineering judgment because information is not available to determine when, or if, a pavement reaches a point when it is no longer feasible to rehabilitate it. The existing slab can either be completely removed and replaced or it can be recycled back into a new rigid or flexible pavement. Before a decision is made to recycle the existing pavement and use the recycled aggregate in a new portland cement concrete pavement or base course, an extensive laboratory testing program should be carried out to demonstrate that the new pavement can be expected not to develop a D-cracking problem throughout its design service life.

Future research needs relative to D-cracking are probably greatest in dealing with pavements that already have developed a D-cracking problem. These include the following:

1. Procedures that can be employed to slow the progression of D-cracking and extend pavement service life,
2. A systematic procedure for evaluating the structural condition and extent of deterioration of a D-cracked slab using nondestructive test methods,
3. Guidelines to determine whether a D-cracked pavement should be rehabilitated or reconstructed, and
4. Rehabilitation alternatives with systematic procedures for selecting the best alternative for a specific project.

CHAPTER ONE

INTRODUCTION

D-cracking, a form of concrete deterioration in portland cement concrete pavement, has been known to exist for approximately 50 years. Within the last several years, however, it has developed into a serious, widespread problem in certain areas of the eastern and west-central United States and in Canada. Appreciable mileages of portland cement concrete pavements have developed D-cracking to various degrees of severity throughout many of the states in a belt extending from northern Texas and Oklahoma northerly and easterly to upper New York State. The reference to portland cement concrete pavement is intended to include pavements made using blended hydraulic cements and blends of portland cement with pozzolans or slag.

The development of D-cracking into a serious problem is a major concern to highway engineers and has generated many questions. These can be summed up in the following three broad questions:

1. How can D-cracking and D-cracking susceptible aggregates be positively identified?
2. How can the development of D-cracking in PCC pavements constructed in the future be prevented or minimized?
3. How can the use of existing pavements that have already developed a D-cracking problem be maximized?

The above questions have been and are being responded to through research efforts by governments, universities, and private industry, and by changes in engineering practice being developed and implemented by the states. The ever-increasing demands being placed on the highway transportation system coupled with limited budgets to carry out these demands dictates that the highway engineer be able to respond to the D-cracking problem as expeditiously as possible. This is especially true with the current emphasis on pavement rehabilitation. In addition, the unprecedented growth in size, weight, and numbers of heavy commercial vehicles since the advent of the Interstate system of highways has placed the engineer in the position of having to rehabilitate pavements to adequately serve under demands far exceeding anything that existed in the past.

This synthesis has been prepared to provide the latest information available on D-cracking to permit the engineer to do a better job of responding to the problem today, tomorrow, and until such time as more knowledge and improved techniques have been developed.

HISTORICAL BACKGROUND OF D-CRACKING

D-cracking was first identified in the 1930s in Kansas. The first research efforts to determine probable causes and solutions were initiated in Missouri in 1940 and in Kansas in 1944. Implementation of the results of the early efforts, which included the elimination of some coarse gravel aggregates for use in portland cement concrete pavement and the reduction in nominal maximum size of coarse aggregate from certain other sources, appeared to greatly reduce the seriousness of the D-cracking problem and little additional work was done over an extended period of time.

During the 1960s and 1970s, following an expanded construction program beginning in the late 1950s, evidence of D-cracking began showing up in increasingly alarming amounts, and D-cracking was again considered a serious problem in several states. Expanded D-cracking research efforts were undertaken in an attempt to develop a better understanding of the problem. Among those who have been very active in D-cracking research are the states of Illinois, Iowa, Kansas, Missouri, and Ohio, and the Portland Cement Association.

Initially, D-cracking was considered a problem associated only with concrete made with crushed-stone coarse aggregate, primarily crushed limestone. Later it was determined that D-cracking also can develop in concrete containing gravel as the coarse aggregate, particularly gravels consisting primarily of particles of sedimentary rock. In some states (e.g., Kansas), D-cracking is primarily associated with crushed limestones; whereas in others (e.g., Illinois), gravel coarse aggregates also are considered a major cause of D-cracking.

DEFINITION AND RECOGNITION OF D-CRACKING

D-cracking is a form of portland cement concrete deterioration associated primarily with the use of coarse aggregates in the concrete that disintegrate when they become saturated and are subjected to repeated cycles of freezing and thawing. It is defined by a characteristic crack pattern that appears at the wearing surface of the pavement as a series of closely spaced fine cracks adjacent and generally parallel to transverse and longitudinal joints and cracks and to the free edges of the pavement slab (*1–4*). D-cracking also is associated with a series of nearly horizontal cracks that characteristically develop in the lower and middle levels of the pavement slab before the appearance of the D-cracking pattern on the wearing surface (*1*). The series of fine, closely spaced parallel cracks that appear at the wearing surface are often filled with a black, blue, gray, or white deposit (*5*) consisting of calcium carbonate and dirt.

Evidence of D-cracking on the pavement wearing surface first appears at the intersections of transverse joints and cracks with longitudinal joints and the free edges of the pavement slab (Fig. 1). The cracking then progresses along the joints and cracks

FIGURE 1 Initial appearance of D-cracking at intersection of transverse and longitudinal joint.

and free edges until it forms a nearly continuous network confined to the peripheral areas of the slab (Fig. 2). If permitted to continue, the cracking will encroach on the remaining central portion of the slab, resulting in complete area distress as shown in Figure 3. In most cases, however, especially in jointed pavement, this advanced stage of total slab deterioration is not observed because the slab is either patched or the pavement is overlaid, thus hiding from view the development of further deterioration.

Information was not found in the literature to explain where the term "D-cracking" originated, or why this type of deterioration has been termed "D-cracking." There is some indication that the name may have originated from the shape of the typical crack pattern. The closely spaced fine parallel cracks tend to fan out at the pavement edges and at longitudinal and transverse joint and crack intersections, thus forming the general shape of the letter D. Some refer to it as "D-line cracking." There are other indications that the name could have stemmed from the durability problem and concrete deterioration that results from the cracking; thus the reference to "D-cracking," "Durability Cracking," or "Deterioration Cracking."

D-cracking originates in the coarse aggregate particles. Certain nondurable aggregates eventually crack when exposed to repeated cycles of freezing and thawing while saturated with water. In time, the cracks in the aggregate particles propagate through the mortar matrix surrounding the aggregate and connect to form a series of microcracks that usually are inclined from the bottom of the slab slightly upward to a joint or crack face. Another series develops in the upper portion of the slab and extends to the pavement wearing surface, there to be observed as the first apparent evidence of D-cracking. In time the microcracks enlarge, resulting in deterioration of the concrete slab.

D-cracking usually originates in the lower portion of the pavement slab and progresses upward (*1*). In some instances, however, it has been known to start at the top of the slab or in the interior (*6*). Even so, it always starts along cracks, joints, and free edges. Because it usually starts at the bottom and progresses upward, deterioration in the bottom of the slab can be fairly extensive before signs of D-cracking show on the pavement surface.

Since its recognition in the 1930s, D-cracking has meant many different things to different observers, ranging from any crack in the concrete pavement that is filled with a deposit at the pavement surface regardless of the cause, to cracks that have resulted from freezing and thawing of concrete whether or not there is subsequent filling of the cracks (*1*). Each and every type of concrete deterioration has at one time or another been identified as D-cracking by some observer.

Over the years there has been confusion between "D-cracking" and "map cracking" in portland cement concrete pave-

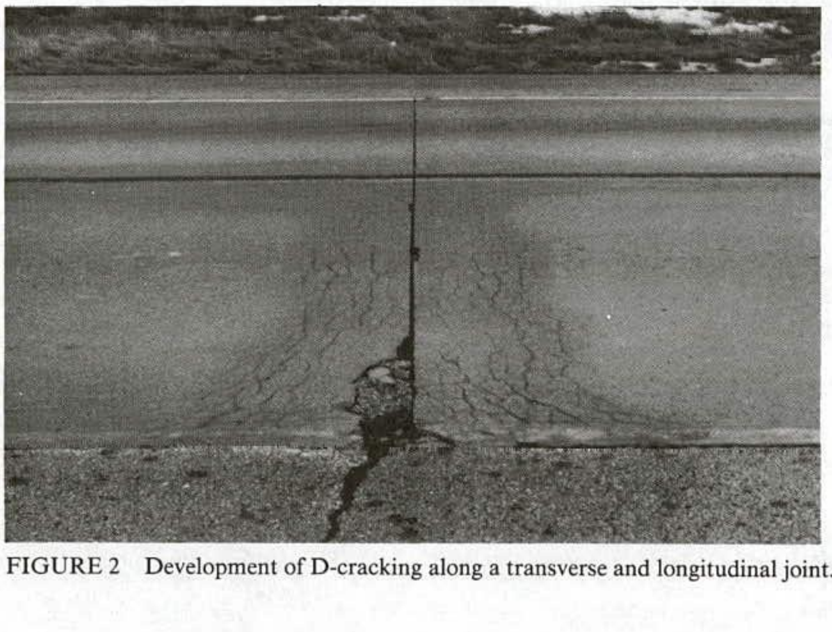


FIGURE 2 Development of D-cracking along a transverse and longitudinal joint.

FIGURE 3 Severe D-cracking affecting most of the pavement slab.

ments. In the earlier days, some considered that the problem was map cracking if the coarse aggregate in the concrete was gravel. It was identified as D-cracking in pavements with crushed-stone coarse aggregate. D-cracking and map cracking, however, are two distinctly different types of concrete failure (*2*) and must not be confused when identifying pavement problems. Although both types of cracking can result in complete disintegration of a slab and both are related primarily to the coarse aggregate used in the concrete, the causes of the problems are different.

Map cracking is typified by many fine interconnecting cracks in the pavement outlining small areas from four to eight inches in their greatest dimension. The general appearance of map cracking is somewhat like that of a heavily lined map. Such failures are often of considerable area and can extend the full width and length of a pavement project (*2*). Map cracking results when the concrete undergoes uniform differential volume change in which the interior becomes larger with respect to the surface or vice versa. An example of the former is alkali-silica reaction map cracking that results from the use of an alkali-reactive coarse aggregate in the concrete (*7, 8*). The alkali-aggregate reaction causes expansion in the concrete and results in concrete cracking. The cracking starts as very fine interconnecting cracks at the pavement surface (Fig. 4) (*9*). The cracks progress ver-

FIGURE 4 Map cracking caused by alkali-aggregate reactivity (*9*).

tically through the slab and in time can become deep, wide, and destructive (*8*). An example of the surface becoming smaller than the interior is drying shrinkage or carbonation shrinkage.

Dubberke and Marks (*10*) report a rapid deterioration similar to D-cracking caused by non-alkali-reactive ferroan dolomites with good pore systems. Concrete beams containing these aggregates exhibit very low expansions and very good durabilities when tested according to ASTM C 666, Method B. The deterioration occurs and spreads from joints and uncontrolled cracks just as in D-cracking, but the pattern cracks on the surface are less parallel and somewhat more widely spaced.

D-cracking quite often can be difficult to positively identify in the field, especially when signs are just beginning to appear on the pavement wearing surface. The first indication of a D-cracking problem can be staining or discoloration of the pavement surface at intersections of transverse and longitudinal joints and along cracks and joints (Fig. 5); this may precede surface cracking (*3, 4, 7*). Although staining is a matter of concern, it is not necessarily an indicator of serious problems to come and cannot be taken as positive indication of a D-cracking problem (*7*).

The only method available today for positive identification of the development of D-cracking in a concrete pavement is examination of full-depth cores taken in the immediate vicinity of joints and cracks (*11*). This method can be used to determine the inception of D-cracking before the appearance of initial signs on the pavement surface. Ohio has reported this method as giving definite results in all cases (*11*). Cores from pavements with early stages of D-cracking are shown in Figures 6 and 7 (*12*). In the first view, only microcracking has developed in the aggregate particles at the bottom of the slab. In Figure 7, the cracking has progressed through the mortar matrix at the bottom. No signs of D-cracking were evident on the pavement surface in either case.

Efforts reported to date have not been successful in finding a nondestructive method for detecting D-cracking before the appearance of initial signs on the pavement surface, or for positive identification after the appearance of initial signs (*11*). Both pulse-velocity measuring equipment and delamination-detection equipment for use on pavement surface were investigated. Also cores were tested for fundamental transverse and longitudinal frequencies. No significant differences could be detected between cores with and without D-cracking in the early stages.

MECHANISMS OF D-CRACKING

Although D-cracking has been around since the 1930s, and fairly early was related primarily to the coarse aggregate in the concrete, it has been an exceedingly difficult problem to solve. Researchers in Kansas, for example, have done considerable research on the problem. As late as 1974 they reported that they finally had been successful in inducing D-cracking in the laboratory, but that the mechanism remains a mystery (*6*).

Today, the overall mechanism by which D-cracking is initiated is believed to be fairly well understood (*1, 9, 12, 13*). D-cracking is caused by disintegration of critically saturated coarse aggregate particles from freezing and thawing. It is initiated when moisture 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 may exceed the internal strength of the aggregate and cause cracking of the aggregate and surrounding mortar. With continued freezing and thawing, existing cracks may provide additional channels for migration of moisture into the slab and also become additional sites for ice formation and the generation of excessive pressures operating to widen the existing cracks. If allowed to

FIGURE 5 Staining or surface discoloration along cracks in a CRC pavement that often precedes surface D-cracking.

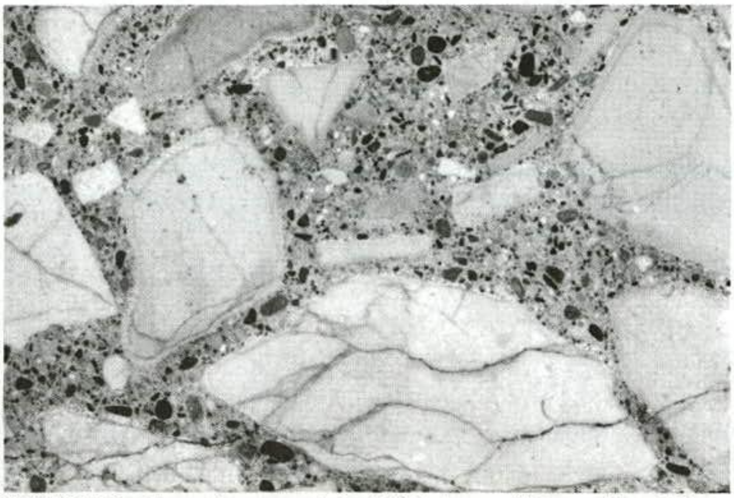


FIGURE 6 Core of pavement in initial stages of D-cracking. Micro-cracking in coarse aggregate particles in bottom of slab, no signs of D-cracking on surface of pavement (*12*).

progress, the entire pavement slab will be converted to a mass of rubble.

The basic mechanism by which the excessive pressures develop in the coarse aggregate particles during cyclic freezing and thawing, however, is not as clearly defined nor as well understood (*14, 15*). Currently there are four theories that have been proposed to describe this basic mechanism (*14*). These include the hydraulic pressure theory proposed by Powers, the desorption theory by Litvan, the diffusion and growth of capillary ice by Powers and Helmuth, and the dual-mechanism theory proposed by Larson and Cady.

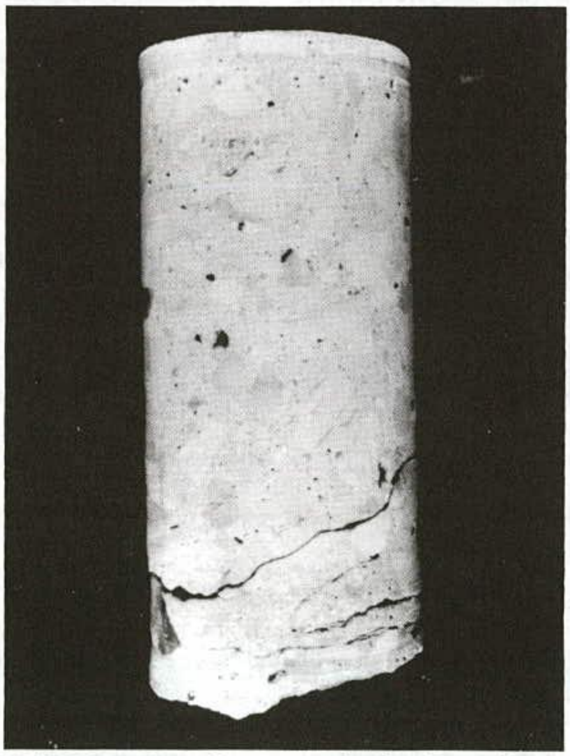


FIGURE 7 Core of pavement showing D-cracking at the lower level with no signs of D-cracking on the pavement surface (*12*).

Powers (*16, 17*) proposed that hydraulic pressure caused by water ahead of an advancing ice front in critically saturated pores could exceed the tensile strength of an aggregate and cause rupturing of the aggregate. As the temperature drops below 32°F (0°C), ice begins to form in the pore spaces. As the temperature continues to decrease, ice will form in progressively smaller pores and displace water that must flow through the unfrozen part of the body to the nearest point of escape. The resistance to this flow of water is termed the hydraulic pressure. The pressure generated is a function of the rate of freezing, the distance the water must travel to escape, the permeability of the body, and the viscosity of the water.

Litvan's theory (*18, 19*) is somewhat analogous to Powers's except that it suggests that differences in vapor pressures force the evacuation of water from the aggregate. It is based on the lowering of relative humidity with decreasing temperature. Migration of water and the resulting dilation caused by hydraulic pressure will begin only when the relative humidity in cement paste reaches the point at which the largest full pore space must be emptied. As cooling progresses, the relative humidity decreases. This decrease is accomplished in part by condensation in the form of ice and partly as a desorption of adsorbed water that then migrates to the nearest point of escape.

Powers and Helmuth (*20*) cited diffusion of water and subsequent growth of ice crystals in capillaries as a disruptive force even at constant temperatures. Differences in free energy between the various phases of water or osmotic potentials caused by differences in solute concentration or both can cause this diffusion to occur.

Larson and Cady (*21*) noted dilation of aggregate particles even after freezing had stopped. In their dual-mechanism theory, they propose that the initial dilation during freezing is caused by hydraulic pressure as suggested by Powers. The post-freezing dilations are attributed to the adsorption of water to ice and rock surfaces. Adsorbed water is ordered water that creates an expansive phase transition the same as the water-to-ice transition.

Although the above four theories differ in their explanations of how pressure develops, they are similar in that all require moisture, cycles of freezing and thawing, and excessive pressure buildup within the coarse aggregate particles to cause D-cracking.

A different type of hypothesis was advanced by Dunn and Hudec (*15, 22*) in 1966 to explain the failure of argillaceous carbonate aggregates. They postulated that the disruptive force is not caused by the expansion of freezing water but results from the expansion of sorbed, ordered water on clay surfaces. With this theory, D-cracking would not be restricted to the winter season but would be a year-round process dependent on the amount of water adsorbed by the coarse aggregate in the concrete. The problem with applying this theory to D-cracking is that D-cracking exists only in areas that have freezing conditions.

FACTORS INFLUENCING THE DEVELOPMENT OF D-CRACKING

Over the years, considerable research has been expended in efforts to isolate those factors influencing the development of D-cracking. Factors that have been evaluated include environmental conditions, coarse aggregate, fine aggregate, cement, pavement design, subsurface drainage, and traffic.

Environmental Conditions

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 use of coarse aggregates that are nondurable when subjected to repeated cycles of freezing and thawing in the continuous presence of moisture. In some of the earlier work in Missouri (*4*), a well-defined trend was found between the variance in freezing cycles from north to south and the intensity of joint deterioration, which pointed toward a correlation with the number of cycles of freezing and thawing. Topography was considered an indirect influence related to drainage. Joint deterioration was greater in the areas of the projects where drainage was poor.

The Portland Cement Association in some work for Ohio (*12*) reported that the physiographic setting appeared to have little bearing on the degree of saturation of pavement concrete, a condition necessary for D-cracking. It made little difference in degree of saturation of the concrete whether it was located on tangent, above or at existing grade, on the crest of a hill, in a low area, under a bridge, or in an open area.

The fact that continuous moisture availability is very important for the development of D-cracking is evident when comparing pavement and bridge decks. Bridge decks do not develop D-cracking, which is attributed to the fact that the underside of a bridge deck slab is not regularly exposed to free water or relative humidities of 100 percent. Thus, the concrete is permitted to dry periodically and the coarse aggregate remains unsaturated (*23*).

Coarse Aggregate

The coarse aggregate in the portland cement concrete is the primary factor influencing the development of D-cracking (*1*–*4*). Aggregates associated with D-cracking have a number of characteristics in common. No single one, however, can be used to satisfactorily predict the inherent durability of the aggregate (*24*).

Composition

Nearly all rock types that are known to be associated with D-cracking are of sedimentary origin, including both carbonate and silicate materials (*24*). They range in composition from essentially pure limestone and dolomite, through those types containing varying amounts of chert and clay, to essentially pure chert, and argillaceous rock types such as shale. Sandstones, graywackes, and other siliceous sedimentary rocks are also known to have caused problems. However, this should not be interpreted to mean that all of these rock types are susceptible to D-cracking. In contrast, aggregates having these same names are known to have performed satisfactorily for more than 25 years.

Materials of igneous origin are not known to be associated with D-cracking (*24*). These include intrusive rock types such as granite, diorite, and gabbro, and extrusive or volcanic materials such as rhyolite, andesite, and basalt.

Materials of metamorphic origin have given variable but generally satisfactory performance (*24*). Gneiss, quartzite, and marble, for example, are not known to be susceptible to D-cracking, whereas rock types such as metagraywacke and a few other partially metamorphosed sedimentary rocks have been known to cause distress.

There has been some work evaluating the D-cracking susceptibility of artificial aggregates (*24*). Thus far, slag aggregates and lightweight materials such as expanded shales and clays have not been found to be associated with D-cracking.

Pore Structure

It is generally accepted that pore structure is the most important characteristic of coarse aggregate influencing its susceptibility to D-cracking. The Portland Cement Association (*9*) reported that the nature of the pore system of the coarse aggregate and temperature variations appeared to be overriding factors determining the degree of saturation of aggregate particles in concrete. When the degree of saturation exceeds the critical value of 0.917, distress in the aggregate particles will develop upon freezing if there are no voids into which excess water can be expelled. The critical value is 0.917 because water increases in volume 9 percent on freezing, a volume change that will take place unless resisted by confining pressures of 30,000 psi (207 MPa) or more.

The characteristics of the pore structure that control just about all pore-related properties are porosity, permeability, and pore-size distribution (*14*). Aggregates with low permeability, high porosity, and small pore size are most vulnerable to freezing problems in concrete (*25*), and thus more susceptible to D-cracking.

The literature suggests that there is a range of pore sizes that can be classified as harmful when aggregates are subjected to repeated cycles of freezing and thawing. Pore sizes below this range are either too small to accept water or so small that water in them does not freeze under normal temperatures. Pores smaller than 25 angstroms (0.25 μm) are considered below the range (*14, 25*). Kaneuji (*25*), in correlating pore-size distribution to frost durability, suggests that the lower limit of the range may include pores as large as 45 angstroms (0.45 μm) in diameter. The upper limit may be as high as 5 μm. A pore diameter of 5 μm has been used to separate "micropores," which are supposedly harmful, from "macropores," which are not (*25*). Pore-size distribution studies have been made on a number of different types of aggregates (*3, 15, 25, 26*), and the pore-size range has varied considerably but is within the limits suggested above. However, Marks and Dubberke (*3*) reported that, with one exception, all nondurable aggregates analyzed had a predominance of pore sizes in the 0.04 to 0.2 μm range; also, aggregates with good to excellent service records do not have a predominance of pore sizes between 0.04 and 0.2 μm.

Sorption

Absorption-adsorption characteristics have revealed differences in the pore properties of durable and nondurable coarse aggregates. Work by the Portland Cement Association for Ohio (*1, 9, 24*) indicates that absorption values, per se, are meaningless above approximately 0.7 percent but, in the same range, adsorption values greater than 0.1 percent identify nondurable materials. Below 0.3 percent absorption, durable materials may show comparatively high adsorption values. Durable materials with high absorption values contain relatively large pore volumes but with pore sizes too large to become critically saturated in pavement exposures. Conversely, durable aggregates with low absorption but high adsorption may become highly saturated but do not fail because they cannot contain sufficient moisture to become overstressed during freezing.

Other Characteristics

Practically all research on D-cracking of portland cement concrete has shown that coarse aggregate particle size can influence its development. The smaller the particle size the better are the chances that the aggregate will be durable. This is discussed in detail in Chapter 2.

There appears to be a correlation between aggregate durability and bulk specific gravity (*4, 15, 25, 27*). The degree of this correlation, however, has not been completely outlined. It has been reported that nondurable aggregates, in addition to having high absorptivities and high degrees of saturation, have low bulk specific gravities (*15, 25*). Indiana (*25*) reported that most of the unsound cherts have bulk specific gravities of less than 2.50 by ASTM Method C 127-39. Researchers in Missouri (*27*) have reported that the gravity gradation (gradation by heavy-liquid separation) of coarse limestone aggregate must be considered to have some degree of influence on its frost resistance. They developed a multiple linear relationship that indicated that limestone coarse aggregate particles having a bulk specific gravity (vacuum saturated, surface dry) below 2.65 reduce the frost resistance of concrete. The results of blending high- and low-specific-gravity aggregates showed a decrease in the resistance of the concrete to freezing and thawing when the percentage of low-specific-gravity stone was increased. In a later study by Missouri (*4*), it was reported that the frost susceptibility of concrete specimens increased as the bulk specific gravity of the limestone coarse aggregate decreased.

There is some information in the literature that for carbonate rocks the insoluble residue content may be related to aggregate durability (*15, 28, 29*). Lemish et al. (*28*) and Hiltrop and Lemish (*29*) in studying Iowa carbonate aggregates concluded that carbonate aggregates with high-insoluble-residue content had poor resistance to freezing and thawing, whereas pure rock types of high carbonate, low-insoluble-residue content and little if any clay had good service records. Rocks with high-insoluble-residue contents were also found to contain large volumes of uniformly small size pores 1 μm or less in diameter, which again indicates the strong influence of pore structure on aggregate durability.

Fine Aggregate

The type of fine aggregate used in the concrete has been shown to have little influence on the resultant resistance to freezing and thawing of the concrete (*4, 12, 30*). Laboratory tests conducted by the Portland Cement Association (*12*) demonstrated that the source of fine aggregate has essentially no bearing on concrete durability, even when derived from coarse aggregate materials of low frost resistance and poor field performance histories.

Initial freeze-thaw testing in Iowa using the ASTM C 666, Method B procedure likewise showed insignificant differences in concrete durability with changes in source of fine aggregate, and a conveniently available local sand was selected as the standard for use in the test. More recent testing after salt treatment of the coarse aggregate before casting the specimens has shown that fine aggregate can affect concrete durability (*30*). A petrographic analysis of the standard sand identified more than 60 percent carbonate particles, many of which are considered nondurable. In comparing the standard sand to a high quality, predominately igneous Mississippi River sand in the freezing and thawing test, the river sand consistently yielded significantly higher durability factors. The high quality river sand has since been adopted as the standard by Iowa for testing purposes in an effort to produce greater consistency in concrete durability testing for evaluating the D-cracking susceptibility of coarse aggregate.

Cement

It is generally agreed that the brand or composition of cement does not significantly influence D-cracking. Likewise, the amount of cement in the concrete as it affects flexural and compressive strengths, and the use of air entrainment do not appear to significantly influence the durability of concrete as it relates to D-cracking.

Missouri (*4*), in its earlier observations, concluded that the brand of cement did not cause a variance in the amount of deterioration and that variations in concrete mixture proportions did not appear to influence the deterioration process. However, in a later study (*31*), it was concluded that a signficant relationship *did* exist between concrete durability, as subjected to laboratory freezing and thawing, and the source of cement. The effect of air entrainment was determined to be insignificant but high-alkali-content cement was determined to be more detrimental than low-alkali cement.

D-cracking studies by the Portland Cement Association for

Ohio (*9*) initially indicated the source of cement to be of minor importance and the level of air entrainment within the existing specified range was found to be essentially of no importance in this problem. Additional laboratory testing provided results that were not totally conclusive. However, it was considered that variations in uniformity in the composition and size of aggregate particles from a given source in individual test specimens precluded determining the relative effect of source of cement on durability. When potentially alkali-reactive aggregates were tested, the alkali content of the cement appeared to be an important factor, as reported by Missouri (*31*).

Kansas (*32*) determined from tests on cores from pavements that D-cracking did not relate to the tensile or compressive strength of the concrete. No relationship was found between the cement content of the concrete and D-cracking or between the water-cement ratio and D-cracking.

Work reported by the Portland Cement Association (*33*) indicated no consistent relationship between the composition and fineness of cements and the behavior of concrete exposed to 20 or more years of freezing and thawing. Normal differences in cement manufacturing procedures and conditions did not significantly affect the durability of the concrete. Air entrainment provided a significant increase in ability of concrete to endure freezing and thawing, particularly in concrete pavements (this is in reference to resistance to scaling but not D-cracking, and the protection is extended to the mortar matrix but not to the coarse aggregate).

Pavement Design

The influence, if any, of pavement design on the development of D-cracking is not clearly defined in the literature. One thing that is clear, however, is that no pavement design has been found to be immune to the problem. This is not at all surprising because D-cracking depends on the nature of the coarse aggregate used in the concrete and its access to moisture during freezing and thawing.

One conclusion that can be drawn from a review of the literature is that the various elements of pavement design are not very significant relative to the development of D-cracking. It can develop whether or not the pavement is reinforced, jointed, or continuous, with or without base, and regardless of the type of base or shoulder used. A second conclusion that can be drawn is that although the various elements of pavement design are not significant to the development of D-cracking, some of them are related to the rate of deterioration that occurs once D-cracking has developed.

Missouri (*4*), in its earlier work, reported that deterioration was found in all pavement designs. Neither uniform nor variable thickness of pavement, bar mat or wire-mesh-reinforcement, or the use or nonuse of subgrade paper prevented deterioration. In some later work (*4, 34*) it was found that pavements with a polyethylene moisture barrier showed staining and D-cracking at an earlier age than did identical pavements without the moisture barrier.

The Portland Cement Association in a controlled field study (*23*) attempted to evaluate the effect of base type on moisture accumulation in concrete slabs, utilizing granular, granular plus a polyethylene moisture barrier, cement-treated, and clay materials beneath the slab. It was determined that gradual increases in moisture content sufficient to critically saturate certain coarse aggregate particles occurred in all concrete slabs regardless of the type of base.

Although D-cracking can develop in both continuously reinforced concrete pavement (CRCP) and in jointed concrete pavement, it is considered potentially much more serious in CRCP. The closely spaced transverse cracks, which are a design feature of CRCP, unfortunately provide numerous paths for moisture movement and accumulation that are necessary for D-cracking. In contrast to jointed pavement where D-cracking usually involves small areas of pavement near joints and transverse cracks, a CRCP can have nearly 100 percent of its area affected by the time signs of D-cracking start showing on the pavement surface.

Subsurface Drainage

The adverse effects of water on overall pavement performance are well documented, and the need for good subsurface drainage cannot be overstressed. It can be exceedingly difficult, however, to support this need on a cost-effective basis. The same can be said for subsurface drainage as it relates to D-cracking. A positive underdrain system is not sufficient to prevent the development of D-cracking, although it may be effective in reducing its rate of development.

Studies of pavement cores by the Portland Cement Association (*1, 9*) revealed no significant difference in the degree of saturation of concrete pavement where the trench containing the longitudinal tile subdrain was backfilled either up to or through the bottom of the base. Thus, although the latter has been found to provide better drainage, it is not adequate to prevent the occurrence of D-cracking. However, the improved drainage facilities did appear to reduce the rate of development of D-cracking.

Pavement observations by the Portland Cement Association (*35*) have indicated that where artificial drains for base drainage have not been installed, conditions are most conducive to the development of D-cracking when a potentially nondurable coarse aggregate has been used. Where longitudinal tile drains have been installed, the rate of development of D-cracking is reduced if the fines in the base do not seal off base moisture migration to the drain inlets.

Traffic

Neither the volume nor the composition of traffic is considered a factor influencing the development of D-cracking but it can accelerate the rate of deterioration of the concrete once D-cracking has developed. Higher volumes and heavier loads can be expected to cause deterioration from D-cracking to progress at a faster rate.

Missouri (*4*) reported that traffic did not appear to be responsible for the occurrence of the deterioration, although it may influence the rate of progression. Kansas (*6*) reported that D-cracking occurs in slabs not subjected to live loads, but seems to progress at a slower rate than it does in slabs that are subjected to live loads.

Kansas (*32*) also reported on observations of a four-lane roadway of identical pavement in which one pavement was subjected to traffic and deicing salts. A portion of the other pavement was closed to traffic without deicing salts for several years. From a distance the unused pavement looked good, whereas the used pavement looked like a never-ending maintenance problem. Close-up observations of the unused pavement, however, revealed the traditional staining and D-cracking patterns, and cores of the pavement verified the observations. Kansas concluded that traffic and deicing salts aid in accelerating the deterioration process from D-cracking but are not necessary for its occurrence.

CHAPTER TWO

COARSE AGGREGATE IN CONCRETE PAVEMENTS

INTRODUCTION

As previously stated in Chapter 1, the development of D-cracking in portland cement concrete pavements is primarily dependent on the pore structure of the coarse aggregate used in the concrete. With this knowledge, the next logical sequence of events in attacking the problem is to determine how to identify D-cracking-susceptible coarse aggregates and what to do with them once they are identified. The next section deals with those tests that can be applied to evaluate the susceptibility of coarse aggregates to D-cracking. The subsequent section addresses possible beneficiation techniques that can be considered in attempting to upgrade D-cracking aggregates to a level acceptable for use in portland cement concrete pavement.

To avoid D-cracking, either the environment must be altered to prevent the coarse aggregate from becoming critically saturated or the aggregate used in the concrete must be inherently durable (*24*). The first alternative has been demonstrated to be infeasible with existing pavement designs; thus the use of inherently durable aggregates appears to be the only approach. The standard tests for aggregate quality, such as the sodium sulfate or magnesium sulfate soundness test (AASHTO T 104), the Los Angeles abrasion test (AASHTO T 96), percent deleterious materials (AASHTO T 11), and specific gravity (AASHTO T 85), that have been used by the states for many years simply are not adequate by themselves to guard against the use of D-cracking coarse aggregate in portland cement concrete for pavements.

NCHRP Report 66 (*21*), published in 1969, included a statement "...it is apparent that no single test having the attributes of speed and simplicity is available or likely to be developed in the immediate future for predicting the performance of coarse aggregate in portland cement concrete that is subjected to freezing and thawing exposure." In this same report it was concluded that the prospects were poor for finding a single quick, simple, economical, and reliable method of identifying frost-susceptible aggregate particles. The degree of saturation and environmental conditions are as important as the deleterious characteristics of the aggregate. Frost susceptibility is therefore meaningless unless it is defined in conjunction with the environmental conditions related to freezing and accessibility to water. The report also concluded that except in extreme cases, rapid tests were not likely to provide a sound basis for accepting or condemning aggregates with respect to their frost susceptibility. The question of whether an aggregate will perform satisfactorily under specific conditions in the field must be answered by field experience or by a laboratory test that closely simulates the anticipated conditions of field exposure.

Although there have been considerable research efforts and advancements made in the state of the knowledge on D-cracking since that time, the above statements and conclusions are as applicable today as they were at the time they were made. Through correlations with field performance histories, laboratory tests have been developed that can satisfactorily predict field behavior and can be used to accept or reject aggregate sources relative to D-cracking susceptibility, but no simple, quick, and economical single test is yet available. The tests that are satisfactory for aggregate source acceptance or rejection are expensive and require extended periods of time to run. The rapid tests are relatively inexpensive but do not provide a sound basis for aggregate control. They can be used, however, in conjunction with the long-time freezing and thawing tests as screening tests, as will be discussed later.

TEST METHODS TO DETERMINE D-CRACKING SUSCEPTIBILITY

Test methods to determine D-cracking susceptibility of coarse aggregate particles can be grouped into two general categories (*14, 21*). Some tests are based on correlations between identifiable and measurable aggregate property characteristics and known field performance of the aggregate in concrete. Others depend on simulating the service environment to which the concrete will be exposed as a method of predicting field performance.

In the environmental simulation category, some of the tests are conducted directly on samples of the aggregate particles whereas others are conducted on concrete specimens containing the coarse aggregate particles. Because coarse aggregate particles behave differently when encased in mortar, tests involving concrete specimens generally have correlated better with field performance records and are considered more desirable for use as a basis for accepting or rejecting aggregate sources relative to D-cracking susceptibility.

A list and descriptions of those test procedures in each of the two test categories that currently appear to offer promise for application to the D-cracking problem are included in the following paragraphs.

Environmental Simulation Tests

Test procedures offering promise for application to the D-cracking problem that simulate the freeze and thaw environment include:

1. Rapid Freezing and Thawing—ASTM C 666 and various modifications

2. Powers Slow Cool—ASTM C 671
3. VPI Single-Cycle Slow-Freeze

Rapid Freezing and Thawing Tests

Rapid freezing and thawing tests are the ones used by most agencies and the ones currently considered as the best for evaluating the frost resistance of coarse aggregate in portland cement concrete. The basic tests are described in ASTM C 666, Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing. There are several other tests being used, most of which represent modifications to ASTM C 666.

ASTM C 666 includes two procedures, Method A for freezing and thawing in water and Method B for freezing in air and thawing in water. The test is designed to accommodate cylinder or prism specimens, although prism specimens are most commonly used. The freezing and thawing cycle consists of lowering the temperature of the specimens from 40° to 0°F (4.4° to −17.8°C) and raising it back to 40°F (4.4°C) in not less than 2 or more than 5 hours. The test involves determination of relative dynamic modulus of elasticity and calculation of durability factor; determination of specimen length change is optional. The determinations are made initially and after each succeeding 36 cycles of freezing and thawing until completion of the test. The test is run through 300 cycles or until the modulus is reduced to 60 percent of the initial modulus (0.10 percent expansion for the optional length change test), whichever occurs first.

The Portland Cement Association (*1, 9, 24*), before its D-cracking studies for Ohio, had been using a rapid freezing and thawing test similar to Method A in its laboratory for many years. Test specimens were 3 × 3 × 11¼-in. concrete prisms that were moist-cured for 14 days before testing at a rate of two cycles per day. Length, weight, and resonant-frequency measurements were made after 5, 14, and 25 cycles and every 25 cycles thereafter to 300 cycles. Failure was normally considered to have occurred in 300 or fewer cycles if the test specimen either reached 0.10 percent expansion, showed a 5 percent weight loss, or the dynamic modulus dropped 30 percent. In correlating test results with field performance for Ohio aggregates, it was determined that length-change measurements were the most meaningful and 350 cycles were required to establish an indisputable separation between Ohio aggregates that had developed D-cracking in less than 15 years and those that had not in more than 15 years. Based on this correlation, the failure criterion for Ohio was established at 0.032 to 0.035 percent expansion in 350 or fewer cycles.

Additional research was conducted by Ohio and reported by Paxton and Feltz (*11*) using ASTM C 666 Methods A and B except that only expansions of the specimens were measured, as with the Portland Cement Association test. They found that 8 cycles per day could be completed using Method A (freezing in water) whereas 12 cycles per day were possible with Method B (freezing in air). When using Method A, problems were encountered with gage pins falling out of the test specimens or the specimens breaking before the testing reached the desired number of cycles. The problems were not always attributable to the coarse aggregate used in the specimens. Also, using Method A there were problems with damage during freezing to the containers holding the specimens and water. For these reasons, it was concluded that Method B was more desirable. Expansions with Method B were smaller but this was overcome by plotting specimen expansion in percent versus number of cycles and computing the area under the curve. As with previous work by the Portland Cement Association, it was determined that it was difficult to differentiate between good and bad aggregate (concerning D-cracking), until at least 350 freeze and thaw cycles had been completed. The failure criterion was set at an area of 2.05. If after 350 cycles of testing the prisms have not expanded such that the area under the curve is more than 2.05, the aggregate being tested is not considered susceptible to D-cracking. A comparison of results with this test and the Portland Cement Association test was made and resulted in nearly identical ranking of coarse aggregate sources relative to D-cracking susceptibility (*36*).

The Missouri freezing and thawing test (*4, 27*) is reported to be similar to ASTM C 666, Method B, except that it includes only one cycle per week consisting of cooling the specimen in a water bath to 40°F (4.4°C) before freezing, a 16-hr freeze at 0°F (−17.8°C), and storing in a saturated lime water solution at room temperature between cycles. Length changes, weight changes, and resonant frequencies (modulus of elasticity) are determined each week. A durability factor is also determined by the equation in ASTM C 666. The test is completed when the dynamic modulus of elasticity drops below 70 percent of its original value (30 percent loss) or at the completion of 100 cycles, whichever comes first. Total permanent dilation (expansion) at the end of the test is not considered a good indicator of D-cracking susceptibility. One study (*27*) indicated that the rate at which the permanent dilations increased over the first 10 cycles appeared to be the best indicator as to the ordering of failure of specimens. Early failure was considered imminent with slope values above 1.5×10^{-4} and no failure in 100 cycles was expected for values of 0.5×10^{-4} or less. Slope values lying between these two values were considered to represent a zone of transition from rapid to slow failures. The Missouri test probably could be best classified as a slow freezing and thawing test rather than a rapid freezing and thawing test.

The rapid freezing and thawing test used by Iowa (*3*) is essentially ASTM C 666, Method B, except that test specimens are first cured for 90 days in the moist room to ensure critical saturation and to allow completion of the major portion of cement hydration to yield the maximum display of subsequent distress of concrete containing nondurable aggregate. Iowa has some cases where service records show rapid deterioration of concrete containing certain ferroan dolomite aggregates with good pore systems on heavily salted roads but relatively good performance with the same aggregate in pavements with limited use of deicing salts. Therefore, a salt treatment test of coarse aggregate was developed for use before freezing and thawing testing (*30*). The salt treatment test as originally developed consisted of five cycles of drying in an oven at 230°F (110°C) for 24 hr, followed by immersion in a 70°F (21.1°C) saturated solution of sodium chloride for 24 hr. The 70°F saturated solution of salt is poured over the aggregate immediately after it is removed from the oven. After salt treatment, the coarse aggregate is rinsed with clean tap water and incorporated into the concrete mixture for the test specimens. The salt treatment has consistently resulted in failure on the rapid freezing and thawing test for ferroan dolomite aggregates. The durability factors of high-quality coarse aggregates with good service records have not been adversely affected by the sodium chloride pre-treatment whereas the durability factors of those aggregates that are adversely affected by salt are reduced to be in better conformity with field performance histories. Iowa has since determined that one cycle of salt treatment is as effective as five, and currently uses only one cycle in its testing.

The Illinois Department of Transportation also uses a modified version of ASTM C 666, Method B (*37*). The modifications include making only measurements of specimen length and computations of length change. The measurements are made at 73°F (22.8°C) instead of at 40°F (4.4°C). Eight cycles of freezing and thawing (0° to 40°F) are completed each day (except on those days that measurements are being made). The freezing and thawing cycle requires 95 min to freeze and 85 min to thaw. The specimens are at or above freezing for 75 min and at or below freezing for 105 min. Based on a comparison of test results with field performance histories, the failure criterion has been established at 0.06 percent expansion in 350 or fewer cycles. Aggregates in specimens that exceed this value are considered susceptible to the development of D-cracking in less than 20 years.

The Kansas Department of Transportation also uses a modified version of ASTM C 666, Method B. Like Iowa, the modification includes revising the specimen curing period to 90 days in the moist room before testing.

Powers Critical Dilation Test

This test is a slow-cooling test currently designated ASTM C 671, Standard Test Method for Critical Dilation of Concrete Specimens Subjected to Freezing. The concrete specimens are 3 × 6-in. cylinders. Specimens of other sizes and shapes may be used, including cores, cubes, or prisms cut from hardened concrete. The specimens are maintained in a constant-temperature bath at a temperature of 35° ± 2°F (1.7° ± 0.9°C). Once every two weeks the specimens are placed in a cooling bath completely immersed in water-saturated kerosene and the temperature is lowered from 35° to 15°F (1.7° to −9.4°C) at the rate of 5° ± 1°F (2.8° ± 0.5°C) per hour. Specimen length changes and cooling bath temperatures are constantly measured during the cooling period, following which the specimens are returned to the constant-temperature water bath. The test continues until the critical dilation is exceeded or until the specimens have been exposed for the period of interest. The number of cycles during which the difference between successive dilations remains constant is designated as the period of frost immunity. The dilation during the last cycle before the dilation begins to increase sharply (by a factor of two or more) is designated as the critical dilation. Highly frost-resistant concrete may never exhibit critical dilation.

ASTM C 682, Evaluation of Frost Resistance of Coarse Aggregates in Air-Entrained Concrete by Critical Dilation Procedures, suggests that the above test may be used for cases where the determination of a period of frost resistance is desirable and no field experience is available for similar aggregates. Work by the Portland Cement Association (*1*) showed that results obtained from rapid freezing and thawing testing correlated better with field performance histories than did the results from the slow-cooling procedure.

Virginia Polytechnic Institute (VPI) Single-Cycle Slow-Freeze Test

Under NCHRP Project 4-3(1) (*14, 21, 38*), this test was developed and evaluated by correlating the results with the durability factor obtained from ASTM C 666, Method A. The single-cycle test involves fabricating 3 × 3 × 16-in. concrete specimens containing coarse aggregates from the sources being evaluated. After curing for seven days, the specimens are placed in a conventional household deep freeze chamber. Strain measurements are made with a Whittemore strain gauge at 5- to 10-min intervals over a 4-hr cooling period. The cumulative length change is then plotted versus time, and the time slope, designated b_t, is determined as the minimum slope that can be found within a one-third-hour or greater range.

A sufficiently strong relationship was found between the time slope, b_t, and the durability factor at 100 freezing and thawing cycles using ASTM C 666, Method A, to indicate that the single-cycle test can be used as a screen test where the durability factor can be considered as a measure of potential field performance. The results have been interpreted as indicating the need for ASTM C 666 to evaluate the durability factor of an aggregate at any time the determined time slope falls within the range of −4.0 to +1.0.

Aggregate Property Characteristics Tests

The procedures that determine aggregate properties are not considered adequate by themselves as a basis for accepting or rejecting aggregate sources relative to D-cracking susceptibility. Some, however, do appear to offer promise for application to the problem as screening tests. These include:

1. Absorption-adsorption test
2. Iowa pore index test
3. Pore size and volume by mercury porisometer

Absorption-Adsorption Test

In an attempt to develop a quick, inexpensive test procedure for evaluating the D-cracking susceptibility of coarse aggregate sources, the Portland Cement Association developed an absorption-adsorption technique (*1, 9, 24*) in which the pore characteristics of the aggregate were compared with pavement service records. The test involves selecting a specimen of the coarse aggregate to be tested, weighing a minimum of 25 g, and sawing the specimen with a water-cooled saw into ¹⁄₁₆- to ⅛-in. thick slices with alternate slices being set aside for absorption and adsorption measurements.

For the absorption measurements, the slices are vacuum-oven dried for 48 hours, then transferred to a desiccator and cooled to room temperature. They are then weighed in a weighing bottle on an analytical balance. Next, the slices are vacuum saturated in a desiccator using boiled demineralized water and, on alternate days, removed, towel dried to the saturated surface-dry condition, and placed in the weighing bottle for reweighing. The procedure is repeated until weight gains are no longer noted. The percent absorption of the test material is determined from the difference between the final and oven-dry weights divided by the oven-dry weight and multiplied by 100.

For the adsorption measurements, the alternate slices are first crushed to the 2.36-mm to 1.18-mm (No. 8 to No. 16) sieve size and then vacuum-oven dried and placed in a weighing bottle of known tare weight, including the stopper. The sample is next cooled to room temperature in a desiccator containing calcium chloride. The bottle with sample is then stoppered and weighed on the analytical balance to obtain the oven-dry weight. The bottle is then opened and placed, together with the stopper, in a desiccator containing a saturated KNO_3 solution that, at 23° ± 1°C, will maintain a relative humidity of 92 percent. After one day of storage and on alternate days, the sample is weighed in the stoppered weighing bottle until weight increases are no longer noted. The percent adsorption is determined by dividing the total weight gain by the oven-dry weight of the sample and multiplying by 100.

A plot of percent adsorption versus percent absorption was developed with a reference curve to separate sound and unsound aggregate, as shown in Figure 8. In addition to identifying D-cracking susceptible coarse aggregates, aggregates that cause pop-outs as well have been identified by test results that generally fall in the upper right hand quarter of the graph in Figure 8.

A more detailed evaluation of the developed absorption-adsorption test by the Portland Cement Association (*9*), as well as evaluations by others (*3, 15, 25*), indicated that the test is too restrictive for application of the results for source acceptance or rejection. It provides an "overkill" in that it will identify some aggregates with satisfactory service records as being potentially nondurable. Although it is not satisfactory for use in acceptance or rejection of coarse aggregate sources, it is satisfactory for use in conjunction with the long-term freezing and thawing tests as a screening test. A change in absorption-adsorption characteristics of aggregate from a given source would be a positive indication of the need to reevaluate the source for acceptance or rejection as coarse aggregate for portland cement concrete pavement.

Iowa Pore Index Test

In recognizing that the D-cracking problem is related to freezing and thawing, and more specifically related to pore size of coarse aggregate, Iowa started an investigation that resulted in the development of the Iowa Pore Index Test (*3, 39*). The objective of the investigation was to develop a test that would

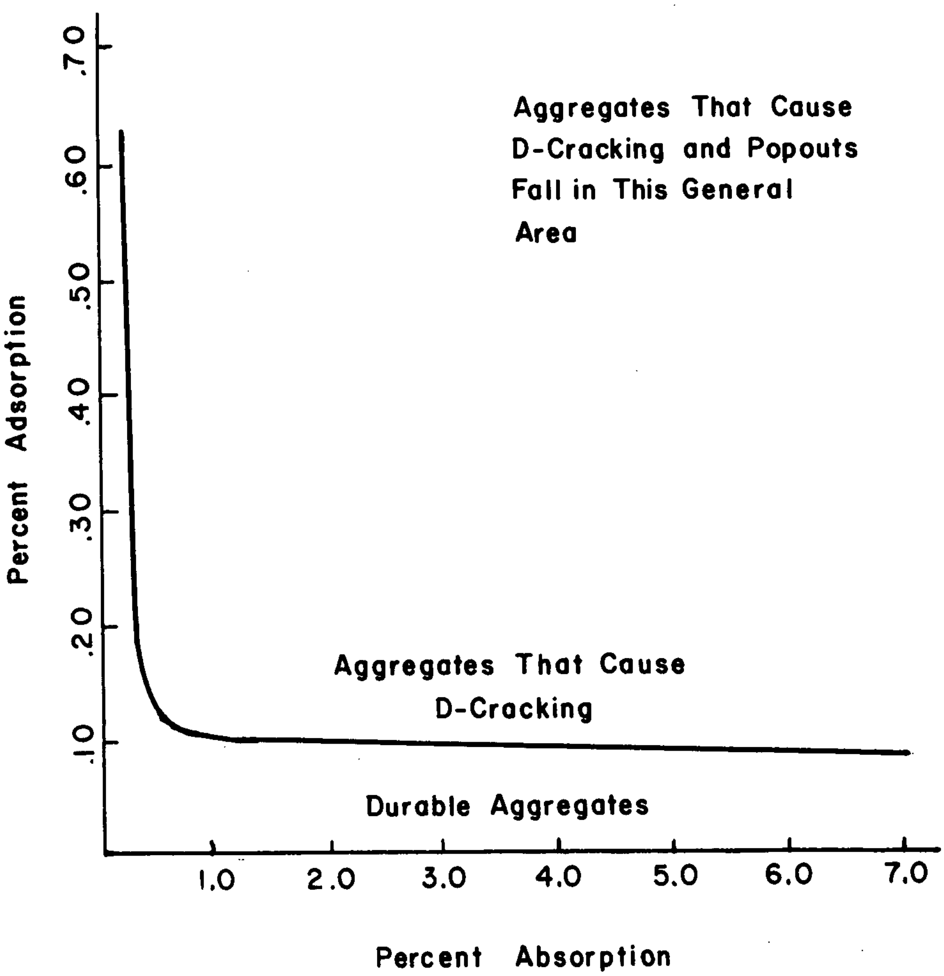


FIGURE 8 Absorption-adsorption data for sound and unsound aggregates (*24*).

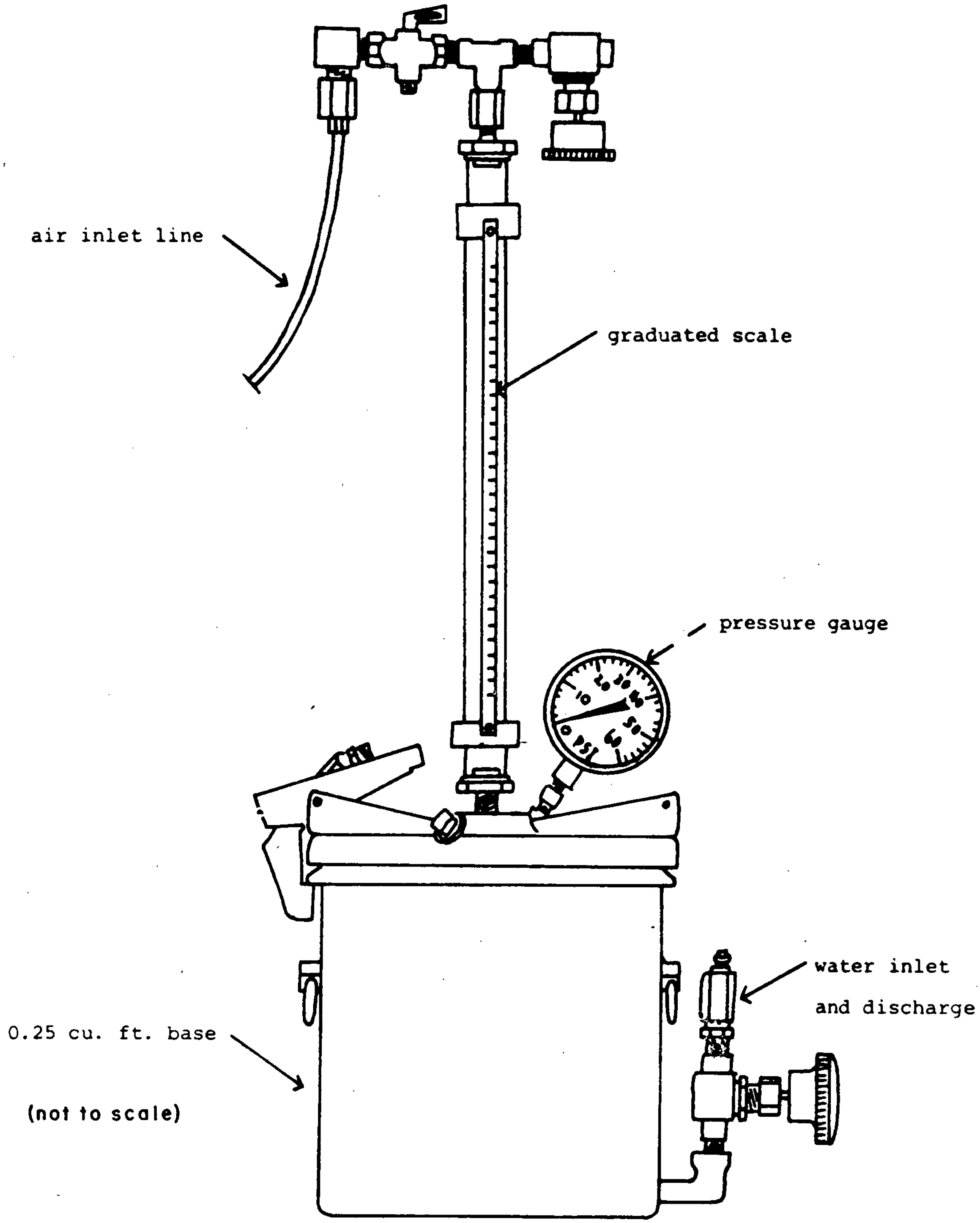


FIGURE 9 Iowa pore index test apparatus (*3*).

readily identify the potential of an aggregate to cause D-cracking because of its susceptibility to critical saturation (*39*) and reduce the time required to conduct the proven freezing and thawing test used by Iowa (5 to 6 months to complete a test).

An air pressure meter modified by replacing the air chamber with a 1-in. diameter by 26-in. long (25 × 660 mm) plastic tube calibrated in millimeters is used to perform the pore index text (see Fig. 9). A 9,000-g sample of oven-dried aggregate cooled to room temperature is placed in the container, the lid is attached, and the testing system is filled with tap water to the "0" mark on the plastic tube. Air pressure at 35 psi (240 kPa) is then introduced at the top of the plastic tube and water level readings are taken at 1 and at 15 min. The 1-min reading represents the amount of water required to fill the macropores of the aggregate and is referred to as the "primary load." A large primary load is considered to be an indication of a beneficial limestone property (*39*).

The difference between the 15-min reading and the 1-min reading represents the amount of water injected into the micropore system of the aggregate and is termed the "secondary load." The secondary load is taken as the "Pore Index" test result.

Sample sizes ranging from 3,000 to 10,000 g have been used, but because the secondary load (Pore Index test result) is directly proportional to the size of the sample, it is necessary to adjust the test results to reflect a 9,000-g sample (the standard sample size).

Iowa's initial work (*39*) suggested a Pore Index value of 27 as the separation between D-cracking and non-D-cracking aggregates. Those with a history of producing D-cracking concrete had Pore Index values greater than 27. In later work (*3*), it was concluded that the Pore Index Test correlates very well with non-argillaceous coarse aggregates, tests of nonuniform materials are not conclusive, homogeneous fractions of coarse aggregates should be evaluated separately if possible, and gravel aggregates should be separated into igneous and carbonate fractions for analysis.

Experience with the Iowa Pore Index Test in Illinois (*37*) identified some trend between field performance and Pore Index for crushed stone aggregates but no discernible trend for gravel aggregates.

Pore Size and Volume by Mercury Porosimeter

As with the above two tests, the application of the results of pore-size distribution by mercury intrusion to the D-cracking problem was investigated in an effort to develop a faster method of predicting the durability of coarse aggregate than that by laboratory freezing and thawing tests. The work was conducted at Purdue University under the Joint Highway Research Project with the Indiana Department of Highways (*25*). Mercury-intrusion porosimetry is considered to be the most accurate and efficient method of measuring pore characteristics (*15*), although it requires expensive equipment and a meticulous procedure. It is based on the fact that the pressure required to force a nonwetting liquid (mercury) into a pore is a function of the surface properties of the liquid and the solid, and the geometry of the pore (*25*).

Mercury porosimeters with a maximum capacity up to 60,000 psi (410 MPa) and capable of measuring pore diameters ranging from 500 μm to 0.25 μm (which covers the entire pore range of interest) are available commercially. The porosimeter consists essentially of a pressure vessel, pressure-generating pump, and measuring instruments for pressure and intrusion.

The amount of sample for the test varies with the porosity of the sample, but usually ranges from 0.5 to 4 g. The test is conducted by selecting a series of pressure points from minimum to maximum, and measuring the intrusion and calculating the pore size and volume at each point. It is necessary that the pressure be held at each point until the intrusion process reaches equilibrium, usually about one minute. Pore-size distribution curves are developed by plotting cumulative pore volume versus pore diameter.

A quantitative correlation between pore-size distribution and the results of the rapid freezing and thawing test, ASTM C 666, Method A, resulted in the following equation:

$$\mathrm{EDF} = \frac{0.579}{\mathrm{PV}} + 6.12\ (\mathrm{MD}) + 3.04$$

where

EDF = Expected Durability Factor,
PV = intruded volume of pores larger than 0.45 μm, and
MD = median diameter of pores larger than 0.45 μm as measured by mercury porosimetry.

In the initial work (*25*), it was concluded that the above equation can be used to predict the frost durability of an aggregate and the following border lines were determined by comparison with field performance data:

EDF	Predicted Durability
Up to 40	Nondurable
40 to 50	Marginal
Over 50	Durable

Additional correlation of the above field performance data (*13*) was made to establish a good-poor EDF dividing line in combination with the maximum amount of poor aggregate that can be included without causing poor pavement performance. The good-poor dividing line was established at an EDF of 50 with no more than 10 percent of the aggregate below this value. In other words, it is considered that 90 percent of the coarse aggregate must have an EDF above 50 if D-cracking is to be avoided.

A further study at Purdue (*15*) showed excellent correlation between results from the mercury porosimeter and the Iowa Pore Index Test. The index test is a quicker, simpler, and more economical procedure, and tests a much larger volume of aggregate. The pore size distribution, however, is not provided by the Pore Index Test.

More recently, Illinois and Indiana conducted an informal study to compare the results obtained by the methods used by the two states to determine the D-cracking susceptibility of coarse aggregate. Samples of Illinois aggregates with known field performances were tested by the rapid freezing and thawing method used by Illinois and by the mercury porosimeter procedure used by Indiana. The D-cracking susceptibility of the aggregate samples as determined by the Indiana method using the mercury porosimeter did not correlate well with the field performance histories.

BENEFICIATION TECHNIQUES TO UPGRADE COARSE AGGREGATE

Beneficiation techniques currently available to upgrade the quality of a D-cracking-susceptible coarse aggregate so that it will be satisfactory for use in portland cement concrete pavements are very limited in number and can produce quite varied results. There are only three techniques being used today to upgrade the quality of coarse aggregate. These include reductions in size of coarse aggregate particles, mechanical separation, and aggregate blending. Reducing the nominal maximum size of coarse aggregate particles is considered to be the best beneficiation technique, whereas aggregate blending is the least desirable. Any one of the three, however, can produce results ranging from completely satisfactory to completely negative relative to the D-cracking problem.

There are two other techniques (*14, 40, 41*) that may offer some potential for consideration in coarse aggregate beneficiation attempts, although they have not yet been used for this purpose. They include aggregate coatings and aggregate impregnation. Ceramic surfaces can be developed on some aggregates by heating them to high temperatures (450° to 800°C). Also, aggregates can be coated by thin films of thermosetting and thermoplastic materials. Linseed oil and polymer impregnation of the coarse aggregate also appears to hold some potential for further development.

An additional beneficiation technique that has been tried in the past without success is drying the coarse aggregate before incorporating it into the concrete. It was considered that this type of preconditioning should at least prolong the time required for the development of D-cracking. Both laboratory results (*24*) and field tests (*32*), however, suggest little or no benefit to be gained by this type of preconditioning.

In considering the application of beneficiation techniques to upgrade a D-cracking-susceptible coarse aggregate, there are two factors that must be evaluated. First, it must be determined that the selected technique will be successful in upgrading the specific coarse aggregate so that it will no longer be susceptible to D-cracking. Second, the cost of upgrading the aggregate should be compared to the cost of transporting durable aggregate not susceptible to D-cracking to the job site. Fortunately, the rapid freezing and thawing tests previously described for coarse aggregate testing for acceptance or rejection are also applicable for evaluating the improvement in aggregate durability resulting from beneficiation efforts.

Reductions in Maximum Size of Coarse Aggregate Particles

The Portland Cement Association (*1, 9, 12, 24, 35*) in its work for Ohio determined that reducing the maximum size of coarse aggregate can significantly increase the resistance to freezing and thawing of aggregates from sources associated with D-cracking. In general, the greatest improvements can be expected from those sources with the poorest service records, with essentially no improvement for the sources that have satisfactory service records (*9*). There are exceptions, however, where reductions in maximum particle size do not improve the durability of poor aggregates and can in some instances make the durability even poorer. It is necessary, therefore, that testing programs be set up to test gravel sources and each ledge or bed from quarries providing coarse aggregate for portland cement concrete pavement to determine what reduction in size, if any, will produce a durable coarse aggregate. A ledge is defined as the projected division of a working face in a quarry. It may contain one or more beds, which is the smallest division of a stratified series and is marked by a more or less well-defined dimensional plane from the rock above or below.

The effects of maximum particle size reduction for carbonate crushed stone coarse aggregate from three sources are depicted in Figure 10. As can be seen, reducing the maximum size from 1½ to ½ in. was not sufficient for the aggregate from Source A to pass the PCA freeze and thaw test. A reduction to 1 in. was sufficient for Source B, whereas a reduction to ½ in. was required to produce a durable aggregate from Source C.

The alternatives for improving the durability of a course aggregate through reduction of maximum particle size are different for gravels and for crushed stones. For gravel aggregates, it appears unavoidable that nondurable material will be incorporated into the aggregate at a given source because of the fine scale on which different rock types occur at the time of initial processing (*24*). Three alternatives are possible for beneficiation through maximum particle size reduction. The first is through the use of naturally occurring finer material by screening and removing the oversize material. The second is by crushing the oversize material and blending it back in. The third is by producing a coarse aggregate composed only of crushed particles by reducing the oversize material. Crushing the oversize and blending it back should produce the greatest quantity of material. The actual alternative that is the best, however, will vary from source to source and must be determined separately for each source. Figure 11 demonstrates the results obtained from maximum particle size reduction through the use of natural and crushed material for three gravel sources of mixed carbonate and silicate aggregate. As shown in Figure 11, the crushed material produced the coarse aggregate most resistant to freezing and thawing in two instances whereas the natural material was most durable in one instance.

For crushed stone material, the only alternative is to further crush the material to a smaller maximum size. The need for maximum particle size reduction, however, must be evaluated on a ledge-by-ledge or bed-by-bed basis and may, or probably will, necessitate the need for selective quarrying, as will be discussed later.

Limited experience in Illinois with crushing oversize gravel suggests that the type of crusher may significantly influence the resistance of the crushed coarse aggregate to D-cracking. In two separate instances where vertical shaft impactors were used to crush oversized gravel from nondurable sources, the resultant crushed gravel passed the Illinois rapid freezing and thawing test easily. In one instance where natural gradations of 1-in., ¾-in., and ½-in. maximum size all failed the test (no more than 0.06 percent expansion in 350 cycles of freezing and thawing), the crushed material was produced in a ½-in. maximum size and passed with expansions ranging from only 0.001 to 0.003 percent. In the other instance, the crushed material was produced in a ¾-in. and a ½-in. maximum size and passed the test with expansions ranging from 0.015 to 0.039 percent, whereas the natural gradations all failed. The current thinking is that, unlike the jaw, cone, and roll crushers that tend to fracture the material in one direction only, the vertical shaft impactor fractures the material along weak seams in any and all directions. Further, it is considered possible that because the aggregate particles are flung outward and contact stationary breaker plates at high impact, the unsound particles completely shatter and are, for the most part, removed from the crushed coarse aggregate. Further verification of these results is needed.

Heavy-Media Separation

Nondurable coarse aggregate can be beneficiated by separation on the basis of specific gravity through heavy-media separation. Heavy liquids are used to float off materials with specific gravities lower than a specified amount. The liquids generally are suspensions of fine, dense material such as powdered mag-

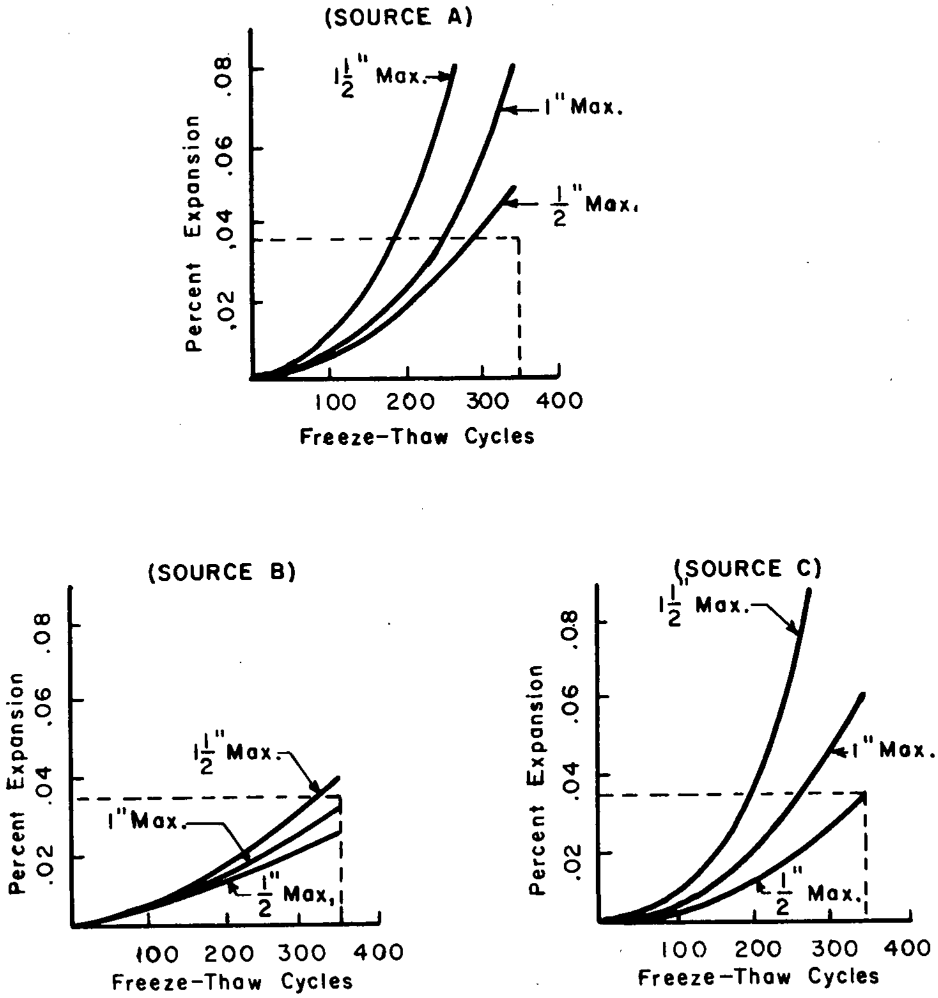


FIGURE 10 Effect of reducing maximum particle size on resistance to freezing and thawing of carbonate aggregates from three crushed stone sources (*24*).

netite and ferrosilicon in water (*14*). By varying the amount of the solids, the specific gravity of the fluid can be varied over a large range.

The assumption behind heavy-media separation is that particles with low specific gravities are less durable than particles with high specific gravities. Although this is certainly an oversimplification of the problem, there is some correlation between D-cracking and specific gravity (*4, 15, 25, 27*). Experience has shown that heavy-media separation does improve the overall durability of an aggregate. It does not provide assurance that the aggregate will not be susceptible to D-cracking, however, and is not considered as reliable as maximum particle size reduction (*24*).

Blending

It is possible to upgrade the quality of an aggregate by blending it with a more durable aggregate. This beneficiation technique simply provides a dilution of nondurable material in the total coarse aggregate gradation with intention of decreasing the concentration of deleterious particles to a level where their effects will be kept to manageable proportions (*14*). The amount of dilution required is somewhat of an unknown factor but can be expected to vary with source and gradation (*24*). Experience in Kansas, where limestone coarse aggregate is primarily responsible for the D-cracking, has shown that pavements with limestone coarse aggregate in excess of 35 percent are more likely to be D-cracked than pavements with less than 35 percent limestone coarse aggregate (*32*).

As previously mentioned, blending is considered the least desirable of the above three beneficiation techniques. For the more extreme durability problems it can only be expected to increase the time required for D-cracking to develop and possibly reduce the severity of distress once it develops. In the case of marginal aggregates, however, it possibly can provide a solution to the D-cracking problem and is worthy of consideration. Again, laboratory freezing and thawing testing can provide a measure of the improvement.

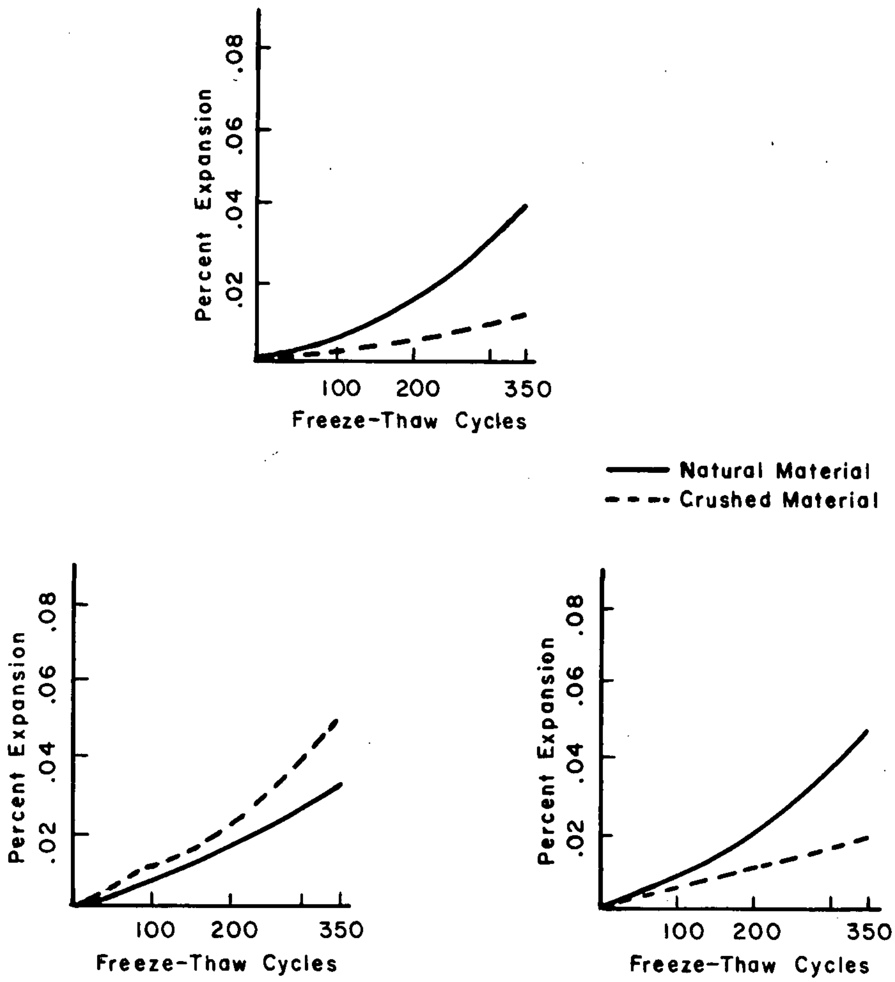


FIGURE 11 Effect of maximum particle size reduction through the use of natural and crushed material from three gravel sources (*24*).

CHAPTER THREE

PREVENTING OR MINIMIZING D-CRACKING IN CONCRETE PAVEMENTS

Preventing the development of D-cracking in portland cement concrete pavement can only be accomplished by eliminating the use of D-cracking susceptible coarse aggregate in the concrete mixture. There are some things that can be done in concrete mixture proportioning and pavement design to minimize the development of D-cracking by slowing its rate of development and possibly reducing the severity of the resultant concrete deterioration, but not to prevent it from occurring.

Preventing the development of D-cracking in portland cement concrete pavement requires carrying out expensive and time-consuming testing programs and tightening materials and construction specifications to guard against the use of D-cracking-susceptible coarse aggregate in the concrete. Also, the application of special considerations in concrete mixture proportioning and pavement design on a project-by-project basis can be of assistance in minimizing the problem.

Much of the information contained in the following sections of the chapter was obtained through on-site visits to several states and discussions with design, construction, materials, and research personnel. The states visited include Illinois, Iowa, Kansas, Missouri, and Ohio.

MATERIALS AND CONSTRUCTION SPECIFICATION REQUIREMENTS

Materials and construction specification requirements as they pertain to D-cracking are primarily concerned with the coarse aggregate used in the concrete and the concrete mixture proportions. Some of the earlier changes in specifications made to attack the D-cracking problem were made on a more or less general basis and limited the maximum size of certain types of aggregate, limited the amount of certain other types that could be present in the total coarse aggregate, and eliminated some types from use entirely. In addition, restrictions were placed on certain aggregate characteristics such as specific gravity, and on types and amounts of cement. Experiences with the early changes suggested that in general they provided help in attacking the D-cracking problem but did not eliminate it. It has since been determined that elimination of the problem requires that the coarse aggregate be tested and approved on a source-by-source basis for gravels and on a ledge-by-ledge or bed-by-bed basis within individual quarries for crushed stones. A very extensive and time-consuming program is required. Further, it is necessary that a combined laboratory and field test program be completed initially to correlate the selected test results with performance histories to establish reliable and realistic failure criteria for material source acceptance or rejection.

Coarse Aggregate

The states that are aggressively attacking the D-cracking problem have set up extensive testing programs for coarse aggregate acceptance; usually on a source basis. Most of them are using environmental simulation type testing, and quite often a version of the rapid freezing and thawing method.

Using a modified version of ASTM C 666, Method B, Illinois has established a rating list indicating whether or not the source is acceptable and what limitations, if any, are placed on the maximum size of the coarse aggregate and the production method. The list identifies each source location and the specific ledge within the source for crushed stones. In testing crushed stones for acceptance, three samples of each gradation being produced are obtained from each ledge to be tested. Three prisms are made from each sample and the test result for a sample is recorded as the average expansion for the three prisms. The sample for a specific gradation is acceptable if the average of the results for the three prisms is not more than 0.06 percent expansion and no individual test is more than 0.075 percent. The source is acceptable for a specific gradation if all three samples pass the failure criteria. For gravels, which are more variable than crushed stones, five samples of each gradation being produced are obtained from each source. If one of the five samples for a specific gradation fails, an additional set of five samples for that gradation is obtained and tested. If all of the additional samples pass, the source is acceptable for the gradation being tested. If one or more fails, the source is rejected for that gradation. The rating list is updated as additional testing is completed. The laboratory is geared to a more or less continual sampling and testing program. Currently, there is no set resampling frequency from a given source as long as the aggregate production is from the same ledge or same general area. Any change in production calls for an immediate reevaluation of the source.

Iowa has established three durability classes of coarse aggregate for use in portland cement concrete pavement based on performance histories, the results of a modified version of ASTM C 666, Method B, and other related aggregate characteristics and tests. Class 1 durability exhibits the poorest durability. Satisfactory performance can be expected for approximately 10 years. Class 2 durability coarse aggregate will produce concrete with an expected service life of more than 10 years, and usually 20 years or more. If a performance history is not available, Class 2 durability coarse aggregate is required to have a Durability Factor of at least 80 by the Iowa freezing and thawing test. Class 3 durability coarse aggregate will produce concrete with an expected service life of more than 20 years, and usually 30 to 35 years. In the absence of a performance history, Class 3 durability coarse aggregate is required to have a rapid freezing and thawing Durability Factor of at least 90. Iowa does not run the rapid freezing and thawing test on gravel aggregates unless they contain a high percentage of limestone particles—in these cases the limestone particles may be removed and tested separately. Only Class 3 durability coarse aggregates are permitted on Interstate highways and thin-bonded PCC overlays. Class 2 durability coarse aggregates are permitted on all other state highways. Class 1 durability coarse aggregates are permitted only on county highways, although many of the counties are starting to use Class 2 durability coarse aggregates.

In some of its earlier attempts to control D-cracking, Kansas revised its specifications to limit the use of limestone coarse aggregate to no more than 30 percent and reduced the maximum size of limestone coarse aggregate to ½ in. The maximum percentage of limestone coarse aggregate was increased to 50 percent when subgrade paper and transverse joint parting plates were used (this proved not to be satisfactory). Kansas has since adopted ASTM C 666, Method B, modified to include a 90-day cure of specimens, as the standard method of test for aggregate source acceptance. Through a correlation of the results of this test with those from the Acid Insoluble Residue test and the 24-hour Water Absorption test, Kansas (*42*) also developed a "Pavement Vulnerability Factor" (PVF) equation as follows:

$$\text{PVF} = \frac{100\text{ A}}{\text{A} + (\text{B}/0.3846)}$$

where

PVF = Pavement Vulnerability Factor,
A = percent by weight of acid insoluble residue, and
B = water absorption (24 hr).

The specification has since been revised to establish two acceptable classes of limestone coarse aggregate for use in concrete pavement. The first is termed Durability Class I and requires completion of ASTM C 666, Method B, with a Durability Factor of 95 or greater and an expansion of 0.025 percent maximum after 300 cycles of freezing and thawing. Also, the aggregate must have a Kansas Freeze-Thaw Soundness (*42*) of 0.90 or greater. The second is termed Durability Class VI and calls for a minimum aggregate Freeze-Thaw Soundness of 0.95, a maximum PVF of 35, and an Acid Insoluble Residue of 3.5 percent maximum. Durability Class VI is used only as an interim approval pending completion of the ASTM C 666, Method B, test. Kansas does not use the rapid freezing and thawing test on gravels. It samples and tests sources only on request by the quarry owners. The sampling and testing is done by individual beds. In addition, three samples are obtained from each construction project and the test results are recorded for future reference. Kansas publishes a list of acceptable materials that includes the quarry identification number, county, legal description, quarry name, bed thickness, and durability class. A more complete listing is published for all sources tested, including those that did not pass. This list also includes the geological classification and the individual test results.

Missouri has attacked the D-cracking susceptible coarse aggregate problem on a county and geologic-formation basis. Performance histories revealed that 39 counties in Missouri have limestone aggregates that can be susceptible to D-cracking. Aggregates from four limestone formations within these counties have no history of D-cracking. The Missouri specification requires that coarse aggregates for concrete pavement from sources within the 39 counties, except from the four acceptable formations, must meet additional gradation and durability requirements. The nominal maximum size is limited to ½ in. Loss by the Missouri Unconfined Alcohol Freeze-Thaw test is limited to 10 percent, the specific gravity cannot be less than 2.58, and the 24-hr absorption cannot be more than 1.5 percent. No gravel coarse aggregates are permitted in concrete for pavements in Missouri.

Ohio uses still a different approach in approving coarse aggregate for PCC pavement. Approval is by final product on a project-by-project basis rather than by source on a ledge-by-ledge or bed-by-bed basis. The specifications permit coarse aggregate gradations ranging from a 1½-in. to ½-in. nominal maximum size. If a gravel or limestone coarse aggregate is used in a size exceeding ½ in., the material is required to come from a source having a performance history demonstrating little or no D-cracking in 15 or more years, or it must be tested in accordance with ASTM C 666, Method B, and the area under the curve obtained by plotting percent expansion versus the number of test cycles is not permitted to exceed 2.05 in 350 or fewer cycles. The contractor for a project is required to submit samples for testing at least 90 days before starting paving operations.

The previous discussions were not intended to cover all procedures being used by the states to eliminate D-cracking, but only to provide a feeling for some of the differences in approaches. As previously stated, information to date indicates that the only way to eliminate the problem is to identify and eliminate from use those coarse aggregates that are causing the problem. From the standpoint of maximizing the use of locally available aggregate and minimizing the cost of production, testing of coarse aggregate from a source through a range of maximum sizes should be the best approach. This will determine the largest maximum size, if any, from that source that is acceptable for use in the concrete. An across-the-board reduction in maximum size of coarse aggregate, on the other hand, can be too restrictive for some sources and unnecessarily increase the cost of production, while not being restrictive enough for others and can permit the use of nondurable aggregate.

Discussions with personnel from several of the states that have included freeze-and-thaw testing in the procedures for acceptance of coarse aggregate for PCC pavement have revealed that the procedures have not been in use long enough to provide positive evidence of their effectiveness in eliminating D-cracking. Opinions, however, are very favorable. Earlier changes that did not include freeze-and-thaw testing and aggregate acceptance on a source or project basis have not eliminated the problem.

Concrete Mixture Proportioning

It appears that little is being done in the area of concrete mixture proportioning as it pertains to D-cracking. As previously mentioned, nothing can be done in this area that will eliminate it, but there are some things that can be done to minimize the problem by delaying its development and possibly reducing the resultant deterioration.

Blending a durable aggregate with a marginal aggregate is one possibility. This has the effect of reducing the amount of nondurable aggregate in the concrete. The amount of blending that is required has not been determined. Work in Iowa has indicated that as little as 15 percent of nondurable coarse aggregate can cause D-cracking. To be effective, blending should be done on a project-by-project basis, and laboratory testing should be performed to determine if blending will eliminate the problem and what percent is necessary. The actual blending should be performed during concrete batching operations to ensure a uniform blend throughout the entire pavement.

At the present time at least, the amount or type of cement does not appear to be a significant factor in minimizing D-cracking. The use of high-alkali cement, however, should be avoided if the coarse aggregate shows indications of being alkali reactive.

Adequate air entrainment is necessary to ensure a durable concrete if the concrete is to be subjected to freezing and thawing while critically saturated with water. The air entrainment contributes to frost resistance of the mortar but does not protect the concrete from D-cracking.

The major change in mixture proportioning relating to D-cracking that has been made is an increase in the fine aggregate content. The increase in fine aggregate content has been made partly to increase workability necessitated by requiring smaller maximum size coarse aggregate. Increasing the fine aggregate content also has the effect of reducing the amount of nondurable material in the concrete should any be present in the coarse aggregate.

PAVEMENT DESIGN

The on-site visits to the five states revealed that no major changes have been made in pavement design in response to the D-cracking problem. Research to date indicates that the various elements of pavement design are not very significant relative to the development of D-cracking and suggest that there are no changes in pavement design that can be expected to eliminate the problem.

In an effort to more clearly define the relationships between pavement design and D-cracking, Missouri and Ohio have included experimental test sections with controlled design and material variables as part of the construction of a regular highway. The Missouri pavement was constructed in 1977, and as of 1986 the condition of the sections had not changed sufficiently that an evaluation could be made. The Ohio test pavements were constructed in 1974 and included pavement type, pavement thickness, joint design, joint sealant, base type, and subsurface drainage as design variables. By 1984, early signs of D-cracking were evident in some sections. An interim report (*43*) covering the first 10 years of pavement performance has been prepared, and the study is being continued to mid 1990.

Pavement Type

Pavement type, whether it is a jointed pavement with or without reinforcement or a continuously reinforced pavement, appears to have little or no influence on the development of D-cracking. Once D-cracking starts to develop, however, the problem can become much more serious if the pavement is continuously reinforced. For this reason, it may be advisable to be more restrictive in the specifications governing the coarse aggregate for this type of pavement. In Illinois, for example, where gravel coarse aggregates are a major cause of the D-cracking problem and are considerably more variable than crushed stone in the rapid freezing and thawing test, gravel is not permitted to be used as the coarse aggregate in the concrete if the pavement is to be continuously reinforced. Additional tentative restrictions also have been placed on crushed limestones from certain sources when used in CRCP. The use of a maximum size one size smaller than that included on the approved list is required for those limestone sources that have passed the rapid freezing and thawing test by only a narrow margin until additional sampling and testing can be completed.

Concern has been expressed that problems with load transfer devices may be contributing to the D-cracking problem at transverse joints in jointed pavement. Dowel-bar corrosion and subsequent working of the bars under traffic may be creating cracks and voids for water to collect and cause D-cracking to start at slab middepth and progress both ways, resulting in excessive deterioration in the immediate vicinity of the joint that often develops much more rapidly than at transverse cracks. This suggests the need for considering the use of corrosion-resistant dowel-bar assemblies.

Differences of opinion exist on the effect of joint sealing on D-cracking. Preliminary results from the Ohio test project (*43*) indicate that after 10 years of service there were no discernible differences in joint performance, whether sealed with neoprene, a hot-pour material, or left unsealed. It has been generally accepted, however, that a properly designed and sealed joint will improve joint performance, and, thus, pavement performance. This, in turn, should help reduce the rate of concrete deterioration in the event that D-cracking develops.

Joint spacing is another area that may need special consideration when attempting to eliminate D-cracking by reducing aggregate particle size. A conclusion in the interim report on the Ohio experimental test project (*43*) is that reducing the maximum particle size of coarse aggregate to eliminate D-cracking may increase the frequency of intermediate transverse cracks in pavement panels, and may increase the severity of faulting at these cracks. The frequency of transverse panel cracks was much greater for the ½-in. maximum size, and the frequency increased progressively with increased joint spacing.

Base

Research and past experience has shown that base and type of base material have little or no bearing on the development of D-cracking in concrete pavement. Whether or not base or base type can influence the severity of the problem once D-cracking has developed has not yet been completely evaluated.

The proper selection of base in pavement design, however, will improve pavement performance by preventing loss of slab support and reducing stresses induced by traffic loadings. Thus, although the use of a base will not eliminate D-cracking, it appears reasonable to expect that its use should minimize the problem by reducing the rate at which deterioration develops.

Drainage

Good drainage, both surface and subsurface, is a prerequisite for good pavement performance. Surface water is believed to be the major source of free water beneath a pavement slab in most instances. Rapid removal of this free water is essential, and can best be accomplished through the installation of a positive drainage system. Longitudinal pipe underdrains are effective in accomplishing this. It is important, however, that they are properly outletted and that the outlets and longitudinal drains are kept open and properly maintained. The newer, flexible, rectangular-shaped, drainage mat underdrains (consisting of a filter envelope permanently bonded with a suitable adhesive to an internal plastic core) are proving to be even more effective than the pipe underdrains in removing water from beneath the pavement surface.

The installation of a positive drainage system in itself will not eliminate D-cracking but it can be expected to help reduce the rate of development and the severity of the resultant concrete deterioration. Inclusion of a positive underdrain system should be considered part of a total package to eliminate D-cracking.

Further support for the above recommendation is offered by Missouri's experience with a vapor barrier placed beneath the pavement to prevent subsurface moisture from penetrating the bottom of the slab. Moisture from surface water was retained by the vapor barrier and D-cracking developed earlier and at a faster rate. Elimination of the vapor barrier and inclusion of a positive underdrain system should eliminate this problem.

CHAPTER FOUR

REHABILITATION AND RECONSTRUCTION TECHNIQUES FOR D-CRACKED PAVEMENTS

Information on what to do with existing pavements that have developed D-cracking is lacking, and probably more so than for any other area of the total D-cracking problem. Research efforts, although very limited, have not established a means of stopping the progression of the D-cracking, once it has started. Discussions with state personnel during the on-site visits revealed that the placement of a bituminous concrete overlay over the existing pavement is the primary method of rehabilitation for D-cracked pavements. The procedures being followed are the same as those for any other pavement, although the need for better procedures is evident. The use of portland cement concrete overlays to rehabilitate D-cracked pavements has been very limited, as has reconstruction of D-cracked slabs by recycling.

The suggestions included in the following discussions of this chapter are based on the best information available. The extent to which they should be considered depends on economics and on the importance of the particular project in question. The most consideration should be given to Interstate projects, next to primary highways, and the least to secondary and local projects.

The rehabilitation of jointed pavement and continuously reinforced concrete pavement are discussed separately because each has its own special problems and separate considerations.

REHABILITATION TECHNIQUES FOR PLAIN AND REINFORCED JOINTED CONCRETE PAVEMENTS

The placement of a bituminous concrete overlay over the existing pavement is the procedure most often used to rehabilitate D-cracked slabs. The overlay will not stop the progression of D-cracking. In fact, some recent work by the University of Illinois for the Illinois Department of Transportation (*44*), involving environmental simulation testing in the laboratory, indicated that overlay thicknesses in the 2- to 4-in. range actually accelerated the damage caused by freezing and thawing. Although bituminous concrete overlays decrease the number of cycles of freezing and thawing, which is beneficial, they also decrease the cooling rate, and the combined effect of the two is reported as being detrimental. They also tend to keep the concrete from drying and thus it will be in a critically saturated state earlier in the winter. Nevertheless, thin bituminous concrete overlays have been shown to be effective in improving the riding quality, reducing maintenance costs, and extending the service lives of D-cracked concrete pavements.

Adequate cleaning and repair of the D-cracked pavement before resurfacing is a prerequisite to obtaining good performance from the overlaid pavement. This is an area of major concern to the states. Full-depth patching, in many instances, has not been extended far enough to remove all deteriorated concrete, or the face of the existing concrete to remain in place was damaged during concrete removal. Also there have been many instances where contracts have had appreciable overruns in patching quantities. All of this illustrates the need for a detailed evaluation of the condition of the existing pavement to be made during the plan preparation stage, with cores being taken to determine the extent of concrete deterioration at joints and cracks. Additional coring may be required during actual patching, or the vertical face of the pavement edge can be exposed in areas to be patched to determine the extent of the D-cracking.

Both partial and full-depth patching is being performed, and the patching materials being used include both asphalt and concrete mixtures. The current trend, especially for Interstate and other heavily traveled highways, is to use portland cement concrete for full-depth patching and to reestablish load transfer. Smooth dowels are either being placed at each end of a patch, or at one end with the other being tied with large (usually No. 8) deformed bars. For longer patches, the joint may be restored at its original location with both ends of the patch being tied to the existing pavement. Restoring the joint at its original location would appear to be the best for D-cracked pavement. The working joint would then be in the new concrete constructed with durable coarse aggregate. The tied ends and any bond that develops between the existing concrete and the patch also should help to deter the progression of D-cracking in the existing pavement adjacent to the patch.

When concrete is used in partial-depth patching, the edges of the patch are squared up by sawing and chipping out with a light chipping hammer. When asphaltic concrete is used, the edges may or may not be squared up. No problems with either method were reported.

Differences in opinion were expressed relative to providing expansion relief in D-cracked pavements before resurfacing. Some states require reestablishment of 4-in. (100 mm) expansion relief joints formed by maintenance forces or the construction of new joints. Others believe that the use of a full-depth asphalt patch across the pavement is a better method if expansion relief is to be provided. The patch will tend to hump during pavement expansion but the ride can be maintained by periodically trimming the patch. Still others believe that overall performance will be better if the compression state of the existing pavement is left unchanged.

Discussions during the on-site visits indicated that the states, for the most part, are not increasing the thickness of overlays because of D-cracking. Some are attempting to resurface a D-cracked pavement as soon as possible and before its condition becomes too bad. Some consideration of an increase in the minimum thickness of an overlay appears to be in order, however, to compensate for the continued loss of slab strength caused by D-cracking.

There has been minimal use, mostly on a trial basis, of the prefabricated membrane systems and the built-up systems of woven fiberglass and polymerized asphalt to cover and seal deteriorated joints and cracks before overlaying. The limited experience to date looks promising and suggests that there may be certain conditions that can be repaired at a reduced cost in lieu of partial-depth or full-depth patching.

The use of portland cement concrete overlays for rehabilitating D-cracked pavement has been very limited. There are three types of concrete overlays—thin bonded, partially bonded, and unbonded. Iowa constructed a 3-in. (75-mm) and a 4-in. (100-mm) thin bonded concrete overlay over a D-cracked jointed pavement on I-80 in 1980 as a demonstration project. The performance after six years of service is good. There have been some punch-through failures believed to have occurred at locations where more extensive repair of the existing pavement should have been made at the time of overlay. The thin bonded overlay was constructed not with the intent of stopping the progression of D-cracking but to increase the effective thickness and structural strength of the slab, and thus reduce the rate of development of further deterioration.

Bonded overlays require meticulous cleaning of the existing pavement surface, application of a bonding medium, careful placement and consolidation of the resurfacing concrete, and thorough protection throughout the curing period (*45*). Joints must be formed in the overlay that coincide with those in the existing pavement. Intermediate cracks in the existing pavement will reflect through the resurfacing. Thorough and complete repair of all deteriorated areas must be accomplished.

Partially bonded overlays do not require meticulous cleaning of the pavement surface (*45*). All that is required is the removal of grease, oil, paint, asphalt, debris, etc., that would prevent natural bonding. Joints should be formed in the overlay directly over or within 12 in. (300 mm) of those in the existing pavement. Thorough and complete repair of all deteriorated areas is required as above.

Unbonded concrete overlays require no cleaning of the pavement surface other than the removal of any loose or foreign material. Spalls, pop-outs, and other surface depressions should be filled with an acceptable material. Otherwise, repairs of distressed areas need not be performed. A bond-breaking agent is required and may consist of a number of materials ranging from plastic or paper to a thin bituminous concrete overlay. The thin overlay appears the best in that it can serve as a leveling course over the existing slab to permit the construction of a uniform-thickness concrete overlay and will eliminate the possibility of mechanical bonding resulting from faults, spalls, and depressions in the existing pavement. Joints in the new overlay are formed within the panels of the existing pavement but never over existing joints. Unlike the bonded and partially bonded overlays, unbonded overlays need not be of the same type pavement as the existing pavement. In other words, the overlay can be jointed or continuously reinforced regardless of the existing pavement type, and joint spacing need not be the same.

A 6-in. (150-mm) thick unbonded, unreinforced jointed overlay over a D-cracked pavement was constructed by Kansas on Route US 24 in 1978 as an experimental pavement. The existing pavement was 21 years old at the time of being overlaid and was badly D-cracked, with many of the joints patched with asphalt. A thin bituminous concrete overlay was first placed over the existing pavement to level it and to serve as a bond breaker. Joints in the new overlay were formed within the panels of the existing pavement. The unbonded overlay is still performing excellently after eight years of service.

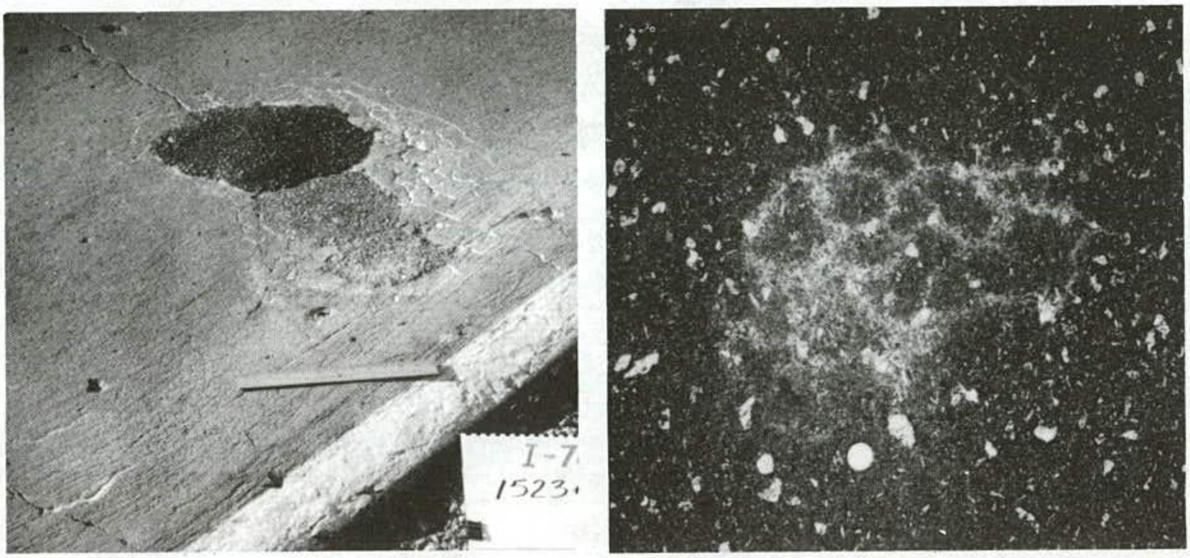
REHABILITATION TECHNIQUES FOR CONTINUOUSLY REINFORCED CONCRETE PAVEMENT

Bituminous concrete overlays have been used almost exclusively for rehabilitating D-cracked continuously reinforced concrete (CRC) pavements. Experience with rehabilitating this type of pavement is much more limited than with jointed pavement and covers only the past six to eight years. Repairs of the deterioration in the existing pavement have been made both with and without retention of the longitudinal reinforcement. In some cases, the patching was done in a manner to purposely destroy the continuity of the reinforcement. The pavement was severed across its entire width at regular intervals and a full-depth bituminous concrete patch was placed to provide for pavement expansion. Both bituminous concrete and portland cement concrete have been used in patching where the continuity of the reinforcement was not retained. Only portland cement concrete is used where the continuity of the reinforcement is being retained.

Performance to date of the overlaid pavements, although generally considered satisfactory, has varied considerably. Where the pavement has been repaired as CRC pavement, reflection through the surfacing of transverse cracks in the existing pavement is almost nonexistent. Where patching has been performed without retaining the continuity of the reinforcement, the extremities of the patch reflect through the new surfacing rather quickly. Wherever the pavement has been purposely severed, wide reflection cracks form, opening as wide as approximately 1 in. (25 mm) during the winter months. The remainder of the pavement has remained relatively free of transverse reflection cracks.

The following recommendations (*46*) should be considered in resurfacing a D-cracked CRC pavement.

1. A D-cracked CRC pavement should be rehabilitated before it requires extensive patching. Rehabilitation costs will be less and performance should be better.
2. Maintain the continuity of the reinforcement in the CRC pavement, if at all possible. This will eliminate most reflective crack problems and improve performance of the rehabilitated pavement.
3. Remove any loose or delaminated concrete on the surface of the pavement and prime and backfill with hot mix, or otherwise square up and repair as a partial-depth concrete patch. Such areas, if not repaired, will reflect through the new overlay fairly quickly and cause failures (*47*). This type of failure is illustrated in Figure 12. Figure 12a depicts a D-cracked distressed area that was not properly cleaned immediately before resurfacing. Figure 12b is the same area taken the following spring, after resurfacing with 2 in. (50 mm) of bituminous concrete. The distress is evident in the new surface.
4. For badly distressed D-cracked pavements, do minimal full-depth patching, very good pavement surface cleaning, and



(a) (b)

FIGURE 12 Effect of improper cleaning of a distressed area before overlaying with bituminous concrete. (*a*) Condition as overlaid in fall, and (*b*) condition following spring.

increase the overlay thickness. This is in support of the hypothesis that the less a badly D-cracked CRC pavement is disturbed by full-depth patching before resurfacing the better are the chances for good performance from the rehabilitation.

Some work in Illinois using heavy woven fiberglass fabric and polymerized asphalt over distressed areas in the CRC pavement before resurfacing looks very promising. Several distressed areas in a 7-in. (175-mm) thick CRC pavement on I-74 were repaired in this manner in 1980 before resurfacing. The areas otherwise would have required a full-depth patch. After six years of service, the repairs are performing excellently with no signs of any problems showing on the overlay surface. The cost of this type of repair is only one-fifth to one-tenth that of a full-depth patch and eliminates the need for disturbing the existing pavement.

Very little use has been made of portland cement concrete overlays in rehabilitating D-cracked CRC pavement. As with the jointed pavement, the Iowa demonstration project constructed on I-80 in 1980 included a 3-in. (75-mm) and a 4-in. (100-mm) thick bonded overlay over an 8-in. (200-mm) thick CRC pavement. Again, the intent was to increase the effective thickness and structural strength of the slab and, thus, reduce the rate of development of further deterioration.

The procedures for constructing a thin bonded, partially bonded, or unbonded portland cement concrete overlay over a CRC pavement are the same as those previously described for jointed pavements. Although not tried, one possible exception could be with the construction of a partially bonded overlay. Because the continuous reinforcement restrains movement in the pavement, it should be feasible to construct a CRC overlay directly over the existing slab without first cleaning and repairing the existing D-cracked pavement.

RECONSTRUCTION TECHNIQUES

When a D-cracked pavement has deteriorated to a condition that it is no longer feasible to rehabilitate it, the only other alternative available to keep the highway open is to reconstruct the pavement. Unfortunately, information is not available to determine when, or if, a pavement reaches this point. Thus, the decision to reconstruct rather than rehabilitate must be made on the basis of economics and engineering judgment.

If a decision is made to reconstruct, the existing pavement can be removed and disposed of, and a new pavement of the same or different type can be constructed in its place, or the pavement can be removed and recycled back into a new rigid or flexible pavement. With the current emphasis on conservation of energy and resources, recycling is gaining in popularity and a number of projects have been carried out, although not many have involved the recycling of a D-cracked pavement.

The first major project was constructed in Minnesota (*48*) in 1980. A 16-mile (26-km) segment of a 25-year-old D-cracked variable-thickness pavement was recycled into a new 8-in. (200-mm) thick non-reinforced pavement.

In 1985, Kansas (*49*) recycled a 2.6-mile (4.2-km) segment of a D-cracked, 25-year-old, 9-in. (230-mm) thick reinforced jointed pavement. The crushed aggregate from the recycled D-cracked pavement was used in all of the portland cement treated base for the new four-lane facility, in a half-mile segment of bituminous paved shoulder, and in a one-mile segment of new 9-in. thick non-reinforced pavement.

Illinois contemplated recycling a badly D-cracked 7-in. (175-mm) continuously reinforced pavement into a new 10-in. (250-mm) thick reinforced jointed pavement. The results of extensive laboratory tests, however, did not provide conclusive evidence that the new pavement could be expected to have a 20-year service life free of D-cracking problems, and plans for the project were dropped.

The remaining few recycling projects utilized the crushed aggregate from the salvaged pavement in base or shoulders, but not in the new pavement. Of course, none of the projects have been in service long enough to provide any information on anticipated performance.

Recycling appears to be very applicable to the reconstruction of D-cracked pavements. Relative to D-cracking, the use of the crushed aggregate from the salvaged pavement in granular or stabilized bases and in bituminous shoulders poses no concern or threat. The potential of future D-cracking when the salvaged material is used in portland cement concrete pavement or base course is a major concern. A decision to use the salvaged aggregate in portland cement concrete pavement or base course should not be made on the assumption that the aggregate has already D-cracked and therefore it will not do it again, or that the reduction in top size from the crushing operation will greatly reduce the coarse aggregate's potential for D-cracking. Rather, extensive laboratory testing should be carried out to demonstrate that the new pavement can be expected not to develop a D-cracking problem throughout its design service life.

Buck (*50*) showed that a concrete made with chert gravel aggregate and having poor resistance to freezing and thawing, even when mature and properly air-entrained (Durability Factor = 3), could be crushed and recycled as aggregate for new portland cement concrete and the durability factor markedly increased (Durability Factor = 23 to 28). The tests were made using ASTM C 666 Procedure A; had Procedure B been used, durability factors would likely have been much higher.

CHAPTER FIVE

ONGOING AND PROPOSED RESEARCH

RESEARCH IN PROGRESS

The following are those formal studies that are known to be ongoing and those that are known to be in the proposal stage with plans being made for future activation. Work in addition to that discussed in the following paragraphs may very well be in progress but not reported in the available information.

The Illinois Department of Transportation has an ongoing study entitled "Geologic Characteristics of Illinois Gravel Deposits Affecting IDOT Freeze-Thaw Results." As previously mentioned, gravels are considered a major source of the D-cracking problem in Illinois. The objective of the study is to evaluate the rock and mineral types in the Illinois gravel deposits and to determine their relationships to the Illinois Freeze-Thaw test. The intent is to develop information that will assist producers in establishing better methods of processing with the hope that the number of approved sources can be increased. The major work on this study is completed and the final report is in the final editing stage. Chert has been identified as the major rock type in Illinois gravels causing loss in freezing and thawing durability. Minor rock types causing freeze and thaw expansion include silty dolomite, ironstone, and possibly weathered carbonate. Sandstone-siltstone, conglomerate, shale, and limestone were considered suspects, but their contents were not high enough in the study samples to cause a significant amount of expansion.

To more clearly define the relationship between pavement design and D-cracking, Ohio constructed in 1974 an experimental pavement project that included pavement type, pavement thickness, joint design, joint sealant, base type, and subsurface drainage as design variables. An interim report (*43*) covering the first ten years of pavement performance has been prepared, and performance evaluations of the test sections are being continued. A discussion of the tentative results is included in Chapter 3 under "Pavement Design."

The Highway Division of the Iowa Department of Transportation has four ongoing projects concerned with D-cracking. The first of these, entitled "Frost Action in Rocks and Concrete," was undertaken to develop new methodology for estimating the frost susceptibility of porous rocks and concrete material using experimental methods for determining expansive pressure, rate of expansion, and pore structure of the rocks and concrete. The work has just been completed and a final report (*51*) released. The study concluded that an ice (phase transition) porosimeter has been fully developed and found to be superior to mercury porosimetry in estimating pore volume because it is capable of measuring body size as well as neck size in determining effective pore size. The occasional presence of globular macropores interconnected by micropores or constrictions appears to be the source of frost damage. In general, the smaller the constrictions or micropores the poorer is the performance. Under severe frost conditions, however, differences in performances between smaller and larger micropores becomes less important. Non-illitic clays in aggregate may behave as micropores of extremely small size and contribute to poor performance.

The second of the Iowa studies is entitled "X-Ray Analysis of Carbonate Aggregate to Predict Concrete Durability." It is being undertaken to determine if a thorough analysis of the pore and chemical properties of an aggregate is sufficient to predict the service life of concrete produced with the aggregate. Testing of various carbonate aggregates before and after treatment with sodium chloride and before and after freezing and thawing testing is being done with x-ray equipment.

The third study in the Iowa program is entitled "Effects of Deicing Salt Compounds on Deterioration of PC Concrete." The objective is to define the deleterious mechanisms resulting from harmful trace compounds introduced into portland cement concrete via deicing salts, to define the extent and economic significance of trace compound poisoning in Iowa, and to determine quantitative salt specification parameters aimed at reducing the harmful influence of deicers.

The title of the fourth study is "Development of a Conductometric Test for Frost Resistance of Concrete." It was undertaken to develop a laboratory test method that would rapidly and accurately predict the performance of concrete subjected to freezing and thawing.

Missouri has four studies in its research program that are related to D-cracking. The first, entitled "Investigation of Roadway Design Variables to Reduce D-Cracking," involves eight experimental sections of pavement constructed on I-35 in 1977. Design variables include two sizes of a coarse aggregate susceptible to D-cracking, pavement constructed with and without a polyethylene moisture barrier, and the use of four different types of base using limestone crushed aggregate. Performance of the test sections is being evaluated. One section using a durable aggregate with no known history of D-cracking has been included as a control section.

A second study in the Missouri series, entitled "Evaluation of Pore Structure in PCCP with Different Maximum Sizes of Coarse Aggregate," was undertaken to determine pore structure parameters of cores from actual pavements with various combinations of mixture proportioning factors. Additional test data such as percent air, slump, compressive strength, and flexural strength obtained under other studies is being made available for comparison.

A third Missouri study is entitled "Influence of Design Characteristics on Concrete Durability" and was undertaken to supplement an ongoing field study of concrete durability constructed on I-435 using three different maximum sizes of nondurable coarse aggregate in five typical mixtures. In addition, a durable aggregate with no known history of D-cracking was included in a control section. The objective of this study is to determine the durability and pore structure parameters of concrete mixed in the laboratory using coarse aggregate and cement from the field study for comparison with field durability.

The fourth study in the Missouri series is concerned with the evaluation of the use of rapid freezing and thawing test equipment and methods for determining concrete durability. Both Method A and Method B of ASTM C 666 will be evaluated and compared with the slow freezing and thawing method currently being used.

SUGGESTIONS FOR FUTURE RESEARCH

The results of past research efforts have provided appreciable information on D-cracking in portland cement concrete pavements that has been of invaluable assistance to the engineer in attempting to deal with the D-cracking problem. Although the complexity of the mechanisms by which D-cracking develops has been outlined, a better understanding is still needed to prevent it from happening in the future. In addition, in the area of pavement management, the smallest amount of effort has been directed at determining the alternatives and best procedures for handling pavements that have already developed a D-cracking problem.

Pre-Implementation Activities for the Strategic Highway Research Program (SHRP) alludes to these needs. Technical Research Area 5 (TRA 5), Cement and Concrete Study, is one of six research areas in the plan and is to cover a five-year period at an estimated cost of $12,000,000. TRA 5 is intended to (a) develop a better understanding of the mechanisms of setting and strength development and the chemical processes during hydration of the cementitious components, and (b) improve the production, placement, quality control, nondestructive testing, and durability of concrete. TRA 5 is divided into four major projects that include (a) Chemistry and Physics of Cement and Concrete, (b) Durability of Concrete, (c) Quality Control and Condition Analysis through Nondestructive Testing, and (d) Mechanical Behavior of Concrete. Although each of the four major projects addresses the D-cracking problem in some manner, the second one, Durability of Concrete, is specifically concerned with it. This project is divided into five tasks. The first task is concerned with minimizing deterioration in existing pavements caused by freezing and thawing, particularly D-cracking and alkali-silica reactivity.

Research directed at providing a more thorough understanding of the mechanisms causing D-cracking should be very helpful. Iowa (*30*) has found some indication that it may result from chemical reactions in some instances in addition to freezing and thawing, especially when deicing agents are used. The complexity of the development of D-cracking is evident when considering it is related primarily to the pore structure of the coarse aggregate but a pore structure analysis by itself will not define the D-cracking susceptibility of an aggregate. There is as yet no simple test to do this although the need still exists. Perhaps, if a thorough understanding of the mechanisms causing D-cracking is achieved, the development of a quick, simple, economical, and reliable test to determine D-cracking susceptibility may be possible.

Additional research in the area of concrete mixture proportioning could be beneficial. The development of a mix with a dense, impermeable mortar may reduce D-cracking. The use of materials such as fly ash, silica fume, ground slag, etc., as replacement for part of the cement or fine aggregate may have a beneficial effect on resistance to freezing and thawing, at least in some instances. More needs to be known about the mechanisms involved.

Beneficiation of coarse aggregate particles by coating has not received much attention. Although generally considered too expensive today, the development of a process for beneficiation by partially drying an aggregate and coating it with a relative inexpensive material such as linseed oil could be practical for application in certain areas where durable aggregates are not available locally and must be shipped long distances.

Relative to the D-cracking problem, the need for better information is probably the greatest in the areas of maintenance and rehabilitation of existing pavements that have D-cracked. Efforts to date to stop D-cracking once it has started have not been successful and probably will not be in the near future. Information on maintenance methods that can be employed to slow the rate of progression and extend the service life is needed. A systematic procedure for evaluating the structural condition and extent of deterioration of a D-cracked slab using nondestructive test methods is needed, as are guidelines to determine whether a D-cracked pavement should be rehabilitated or be removed and replaced. Rehabilitation alternatives with systematic procedures for selecting the best method for a specific project are needed. In other words, procedures are needed that will permit custom designing pavement rehabilitation on a project-by-project basis to maximize the return on the dollar invested.

REFERENCES

1. Verbeck, G., P. Klieger, D. Stark, and W. Teske, "D-Cracking of Concrete Pavements in Ohio," 1st Interim Report, Report No. 72-04, Ohio Department of Highways, Columbus, Ohio (1972) 132 pp.
2. White, V. L. and R. L. Peyton, "Condition of Concrete Pavements in Kansas as Affected by Coarse Aggregate," Highway Research Board Proceedings, Vol. 25, Highway Research Board, National Research Council, Washington, D.C. (1945) pp. 129–146.
3. Marks, V. J. and W. Dubberke, "Durability of Concrete and the Iowa Pore Index Test," in *Transportation Research Record 853: Concrete Analysis and Deterioration,* Transportation Research Board, National Research Council, Washington, D.C. (1982) pp. 25–30.
4. Girard, R. J., E. W. Myers, G. D. Manchester, and W. L. Trimm, "D-Cracking: Pavement Design and Construction Variables," in *Transportation Research Record 853: Concrete Analysis and Deterioration,* Transportation Research Board, National Research Council, Washington, D.C. (1982) pp. 1–9.
5. Bukovatz, J. E., C. F. Crumpton, and H. E. Worley, "Study of D-Cracking in Portland Cement Concrete Pavements, Report 7, Field Phase," State Highway Commission of Kansas, Department of Planning and Development, Research Division (1973) 30 pp.
6. Best, C. H., "D-Cracking in PCC Pavements—Cause and Prevention," Report No. KSU-EES-2472, State Highway Commission of Kansas (1974) 111 pp.
7. Crumpton, C. F. and J. E. Bukovatz, "Map Cracking in Limestone-Sweetened Concrete Pavement Promotes D-Cracking," in *Transportation Research Record 651: Concrete, Aggregates, Marking Materials, Corrosion, and Joint Seals,* Transportation Research Board, National Research Council, Washington, D.C. (1977) pp. 29–31.
8. Ryell, J., B. Chojnacki, G. Woda, and Z. D. Koniuszy, "The Uhthoff Quarry Alkali-Carbonate Rock Reaction: A Laboratory and Field Performance Study," in *Transportation Research Record 525: Cement-Aggregate Reactions,* Transportation Research Board, National Research Council, Washington, D.C. (1974) pp. 43–54.
9. Klieger, P., G. Monfore,. D. Stark, and W. Teske, "D-Cracking of Concrete Pavements in Ohio," Final Report, Report No. Ohio-DOT-11-74, Portland Cement Association, Skokie, Ill. (October 1974) 182 pp.
10. Dubberke, W. and V. J. Marks, "The Relationship of Ferroan Dolomite Aggregate to Rapid Concrete Deterioration," Final Report, Iowa Highway Research Board, Research Project HR-266, Iowa DOT, Ames, Iowa (January 1987) 29 pp.
11. Paxton, J. T. and W. R. Feltz, "Development of Laboratory and Field Methods for Detecting D-Cracking Susceptibility of Ohio Coarse Aggregates in Concrete Pavements," Report No. FHWA/OH/79/006, Ohio Department of Transportation, Bureau of Tests, Columbus, Ohio (1979) 96 pp.
12. Klieger, P., D. Stark, and W. Teske, "The Influence of Environment and Materials on D-Cracking," Final Report, Report No. Ohio-DOT-06-78, Portland Cement Association, Skokie, Ill. (October 1978) 108 pp.
13. Winslow, D. N., M. K. Lindgren, and W. L. Dolch, "Relation between Pavement D-Cracking and Coarse-Aggregate Pore Structure," in *Transportation Research Record 853: Concrete Analysis and Deterioration,* Transportation Research Board, National Research Council, Washington, D.C. (1982) pp. 17–20.
14. Thompson, S. R., M. P. Olsen, and B. J. Dempsey, "Synthesis Report: D-Cracking in Portland Cement Concrete Pavements," Report No. FHWA/IL/UI-187, University of Illinois, Urbana, Ill. (June 1980) 98 pp.
15. Shakoor, A., "Evaluation of Methods for Predicting Durability Characteristics of Argillaceous Carbonate Rock," Ph.D. thesis, Purdue University, West Lafayette, Ind. (April 1982) 252 pp.
16. Powers, T. C., "A Working Hypothesis for Further Studies of Frost Resistance of Concrete," *Proceedings,* American Concrete Institute, Vol. 41 (1945) pp. 245–272.
17. Powers, T. C., "Basic Considerations Pertaining to Freezing and Thawing Tests," *Proceedings,* American Society for Testing Materials, Vol. 55 (1955) pp. 1132–1155.
18. Litvan, G. G., "Phase Transitions of Adsorbates: IV, Mechanisms of Frost Action in Hardened Cement Paste," *American Ceramic Society Journal,* Vol. 55, No. 1 (1972) pp. 38–42.
19. Litvan, G. G., "Adsorptive Systems at Temperatures below the Freezing Point of the Adsorptive," *Advances in Colloid and Interface Science,* Vol. 9 (1978) pp. 253–302.
20. Powers, T. C. and P. A. Helmuth, "Theory of Volume Changes in Hardened Portland Cement Paste during Freezing," Proceedings, Vol. 32, Highway Research Board, National Research Council, Washington, D.C. (1953) pp. 285–297.
21. Larson, T. D. and P. D. Cady, *NCHRP Report 66: Identification of Frost Susceptible Particles in Concrete Aggregates,* Highway Research Board, National Research Council, Washington, D.C. (1969) 62 pp.
22. Dunn, J. R. and P. P. Hudec, "Frost and Sorption Effects in Agrillaceous Rocks," in *Highway Research Record 393: Frost Action in Soils,* Highway Research Board, National Research Council, Washington, D.C. (1972) pp. 65–78.
23. Stark, D., "Field and Laboratory Studies of the Effects of Subbase Type on the Development of D-Cracking," in *Highway Research Record 342: Environmental Effects on Concrete,* Highway Research Board, National Research Council, Washington, D.C. (1970) pp. 27–38.
24. Stark, D., "Characteristics and Utilization of Coarse Ag-

gregates Associated with D-Cracking," Research and Development Bulletin RD047.01p, Portland Cement Association, Skokie, Ill. (1976) 7 pp.

25. Kaneuji, M., "Correlation between Pore Size Distribution and Freeze-Thaw Durability of Coarse Aggregate in Concrete," Report No. JHRP-78-15, Purdue University, West Lafayette, Ind. (August 1978) 142 pp.
26. Kneller, W. A., H. F. Kriege, E. L. Saxer, J. T. Wilband, and T. J. Rohrbacher, "The Properties and Recognition of Deleterious Cherts Which Occur in Aggregate Used by Ohio Concrete Producers," Final Report No. 1014, Research Foundation, The University of Toledo, Toledo, Ohio (November 1968) 272 pp.
27. Missouri State Highway Department, "Investigation of D-Cracking in PCC Pavements: Laboratory Phase 1," Report 71-3, Division of Materials and Research, Jefferson City, Mo. (May 1971) 101 pp.
28. Lemish, J., F. E. Rush, and C. L. Hiltrop, "Relationship of Physical Properties of Some Iowa Carbonate Aggregates as Related to Durability of Concrete," Highway Research Board Bulletin 196, Highway Research Board, National Research Council, Washington, D.C. (1958) pp. 1–6.
29. Hiltrop, C. L. and J. Lemish, "Relationship of Pore Size Distribution and Other Rock Properties to Serviceability of Some Carbonate Aggregates," Highway Research Board Bulletin 239, Highway Research Board, National Research Council, Washington, D.C. (1960) pp. 1–23.
30. Dubberke, W. and V. J. Marks, "The Effect of Deicing Salt on Aggregate Durability," Progress Report for Iowa Highway Research Board, Report 72-4, Iowa Department of Transportation, Ames, Iowa (January 1985) 23 pp.
31. Missouri State Highway Department, "Investigations of 'D' Cracking in PCC Pavements: Phase 2," Report 72-4, Division of Materials and Research, Jefferson City, Mo. (March 1972) 49 pp.
32. Bukovatz, J. E. and C. F. Crumpton, "Study of D-Cracking in Portland Cement Concrete Pavements: Vol. 2—Laboratory Phase," Report No. FHWA-KS-81-2, Kansas Department of Transportation, Planning and Development Department, Topeka, Kan. (1981) 70 pp.
33. Portland Cement Association, "Twenty-Year Report on the Long-Time Study of Cement Performance in Concrete," Research Department Bulletin 175, Research and Development Laboratories, Skokie, Ill. (May 1965) 36 pp.
34. Missouri State Highway Department, "Investigation of 'D' Cracking in PCC Pavements: Phase 3," Report 77-5, Division of Materials and Research, Jefferson City, Mo. (January 1977) 83 pp.
35. Stark, D. and P. Klieger, "Effect of Maximum Size of Coarse Aggregate on D-Cracking in Concrete Pavements," in *Highway Research Record 441: Grading of Concrete Aggregates,* Highway Research Board, National Research Council, Washington, D.C. (1973) pp. 33–43.
36. Paxton, J. T., "Ohio Aggregate and Concrete Testing to Determine D-Cracking Susceptibility," in *Transportation Research Record 853: Concrete Analysis and Deterioration,* Transportation Research Board, National Research Council, Washington, D.C. (1982) pp. 20–24.
37. Traylor, M. L., "Efforts to Eliminate D-Cracking in Illinois," in *Transportation Research Record 853: Concrete Analysis and Deterioration,* Transportation Research Board, National Research Council, Washington, D.C. (1982) pp. 9–14.
38. Walker, R. D., H. J. Pence, W. H. Hazlett, and W. J. Ong, *NCHRP Report 65: One-Cycle Slow-Freeze Test for Evaluating Aggregate Performance in Frozen Concrete,* Transportation Research Board, National Research Council, Washington, D.C. (1969) 21 pp.
39. Myers, J. D. and W. Dubberke, "Iowa Pore Index Text," Interim Report, Office of Materials, Iowa Department of Transportation, Ames, Iowa (January 1980) 23 pp.
40. Marek, C. R., M. Herrin, C. E. Kesler, and E. J. Barenberg, *NCHRP Report 135: Promising Replacements for Conventional Aggregates for Highway Use,* Transportation Research Board, National Research Council, Washington, D.C. (1972) 53 pp.
41. Cady, P. D., P. R. Blankenhorn, D. E. Kline, and D. A. Anderson, *NCHRP Report 207: Upgrading of Low-Quality Aggregates for PCC and Bituminous Pavements,* Transportation Research Board, National Research Council, Washington, D.C. (1979) 91 pp.
42. Wallace, H. E. and L. D. Hamilton, "An Investigation of Kansas Limestones as They Pertain to the D-Cracking Phenomena," prepared for the Region 7 D-Cracking Workshop, February 17 and 18, 1982, Kansas Department of Transportation, Topeka, Kan. (1982) 19 pp.
43. Stark, D., "The Significance of Pavement Design and Materials in D-Cracking," Interim Report No. FHWA/OH-86-008, Ohio Department of Transportation, Columbus, Ohio (March 1986) 82 pp.
44. Jansen, D. J., B. J. Dempsey, J. B. DuBose, and A. J. Patel, "Predicting the Progression of D-Cracking," Report No. FHWA/IL/UI-211, Final Report, University of Illinois, Urbana, Ill. (February 1986) 196 pp.
45. Hutchinson, R. L., *NCHRP Synthesis of Highway Practice 99: Resurfacing with Portland Cement Concrete,* Transportation Research Board, National Research Council, Washington, D.C. (December 1982) 90 pp.
46. Schwartz, D. R., "Continuously Reinforced Concrete Pavement Rehabilitation in Illinois," Proceedings, Pavement Rehabilitation Conference, Oklahoma City, Federal Highway Administration (May 1983) pp. 459–480.
47. Illinois Department of Transportation, "Staged Rehabilitation of CRC Pavement with D-Cracking," Bureau of Materials and Physical Research, Illinois DOT, Springfield, Ill. (March 1983) 12 pp.
48. Halverson, A. D., "Recycling Portland Cement Concrete Pavements," Demonstration Project 47, Minnesota Project, Interim Report No. FHWA-DP-47-3, Federal Highway Administration, Washington, D.C. (May 1981) 66 pp.
49. Fager, G. G., "Recycled Portland Cement Concrete Pavement," Johnson County, Kansas Project No. 7-46-K-0449-02, Interim Report No. FHWA-DP-47-6, Kansas Department of Transportation, Topeka, Kan. (May 1986) 20 pp.
50. Buck, A. D., "Recycled Concrete," in *Highway Research Record 430: Utilization of Waste Materials and Upgrading of Low-Quality Aggregates,* Highway Research Board, National Research Council, Washington, D.C. (1973) pp. 1–8.
51. Enustun, B. V., B. Gunnink, W. Dubberke, and T. Demirel, "Frost Action in Rocks and Concrete," Final Report, Report No. ISU-ERI-86-445, Iowa State University, Ames, Iowa (April 30, 1986) 69 pp.

THE TRANSPORTATION RESEARCH BOARD is a unit of the National Research Council, which serves the National Academy of Sciences and the National Academy of Engineering. It evolved in 1974 from the Highway Research Board, which was established in 1920. The TRB incorporates all former HRB activities and also performs additional functions under a broader scope involving all modes of transportation and the interactions of transportation with society. The Board's purpose is to stimulate research concerning the nature and performance of transportation systems, to disseminate information that the research produces, and to encourage the application of appropriate research findings. The Board's program is carried out by more than 270 committees, task forces, and panels composed of more than 3,300 administrators, engineers, social scientists, attorneys, educators, and others concerned with transportation; they serve without compensation. The program is supported by state transportation and highway departments, the modal administrations of the U.S. Department of Transportation, the Association of American Railroads, the National Highway Traffic Safety Administration, and other organizations and individuals interested in the development of transportation.

The National Academy of Sciences is a private, nonprofit, self-perpetuating society of distinguished scholars engaged in scientific and engineering research, dedicated to the furtherance of science and technology and to their use for the general welfare. Upon the authority of the charter granted to it by the Congress in 1863, the Academy has a mandate that requires it to advise the federal government on scientific and technical matters. Dr. Frank Press is president of the National Academy of Sciences.

The National Academy of Engineering was established in 1964, under the charter of the National Academy of Sciences, as a parallel organization of outstanding engineers. It is autonomous in its administration and in the selection of its members, sharing with the National Academy of Sciences the responsibility for advising the federal government. The National Academy of Engineering also sponsors engineering programs aimed at meeting national needs, encourages education and research, and recognizes the superior achievements of engineers. Dr. Robert M. White is president of the National Academy of Engineering.

The Institute of Medicine was established in 1970 by the National Academy of Sciences to secure the services of eminent members of appropriate professions in the examination of policy matters pertaining to the health of the public. The Institute acts under the responsibility given to the National Academy of Sciences by its congressional charter to be an adviser to the federal government and, upon its own initiative, to identify issues of medical care, research, and education. Dr. Samuel O. Thier is president of the Institute of Medicine.

The National Research Council was organized by the National Academy of Sciences in 1916 to associate the broad community of science and technology with the Academy's purposes of furthering knowledge and advising the federal government. Functioning in accordance with general policies determined by the Academy, the Council has become the principal operating agency of both the National Academy of Sciences and the National Academy of Engineering in providing services to the government, the public, and the scientific and engineering communities. The Council is administered jointly by both Academies and the Institute of Medicine. Dr. Frank Press and Dr. Robert M. White are chairman and vice chairman, respectively, of the National Research Council.

TRANSPORTATION RESEARCH BOARD
National Research Council
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

ADDRESS CORRECTION REQUESTED

NON-PROFIT ORG.
U.S. POSTAGE
PAID
WASHINGTON, D.C.
PERMIT NO. 8970

000015M001
JAMES W HILL
RESEARCH SUPERVISOR
IDAHO TRANS DEPT DIV OF HWYS
P O BOX 7129 3311 W STATE ST
BOISE ID 83707