
TRANSPORTATION RESEARCH RECORD

525

Formerly issued as Highway Research Record

Cement-Aggregate Reactions

7 reports

TNRB

TRANSPORTATION
RESEARCH BOARD

NATIONAL RESEARCH
COUNCIL

Washington, D. C., 1974

PB 242 422

Transportation Research Record 525
Price \$3.00

subject areas

32 cement and concrete

35 mineral aggregates

Transportation Research Board publications are available by ordering directly from the Board. They are also obtainable on a regular basis through organizational or individual supporting membership in the Board; members or library subscribers are eligible for substantial discounts. For further information, write to the Transportation Research Board, National Academy of Sciences, 2101 Constitution Avenue, N.W., Washington, D.C. 20418.

These papers report research work of the authors that was done at institutions named by the authors. The papers were offered to the Transportation Research Board of the National Research Council for publication and are published here in the interest of the dissemination of information from research, one of the major functions of the Transportation Research Board.

Before publication, each paper was reviewed by members of the TRB committee named as its sponsor and accepted as objective, useful, and suitable for publication by the National Research Council. The members of the review committee were chosen for recognized scholarly competence and with due consideration for the balance of disciplines appropriate to the subject concerned.

Responsibility for the publication of these reports rests with the sponsoring committee. However, the opinions and conclusions expressed in the reports are those of the individual authors and not necessarily those of the sponsoring committee, the Transportation Research Board, or the National Research Council.

Each report is reviewed and processed according to the procedures established and monitored by the Report Review Committee of the National Academy of Sciences. Distribution of the report is approved by the President of the Academy upon satisfactory completion of the review process.

LIBRARY OF CONGRESS CATALOGING IN PUBLICATION DATA

National Research Council. Highway Research Board.

Cement-aggregate reactions: 7 reports.

(Transportation research record; 525)

Prepared for the 53d Annual Meeting of the Highway Research Board.

1. Alkali-aggregate reactions—Addresses, essays, lectures. 2. Concrete—Deterioration—Addresses, essays, lectures. I. National Research Council. Transportation Research Board. II. Title. III. Series.

TE7.H5 no 525 [TA439] 380.5'08s [620.1'36'2] 75-15810

ISBN 0-309-02372-6

CONTENTS

FOREWORD	v
KANSAS CONCRETE PAVEMENT PERFORMANCE AS RELATED TO D-CRACKING John E. Bukovatz, Carl F. Crumpton, and Herbert E. Worley	1
EXPANSION OF REACTIVE CARBONATE ROCKS UNDER RESTRAINT M. H. Hilton	9
15 YEARS OF LIVING AT KINGSTON WITH A REACTIVE CARBONATE ROCK P. Smith	23
REACTION PRODUCTS IN EXPANSION TEST SPECIMENS OF CARBONATE AGGREGATE Hollis N. Walker	28
DEVELOPMENTS IN SPECIFICATION AND CONTROL Bryant Mather	38
THE UHTHOFF QUARRY ALKALI-CARBONATE ROCK REACTION: A LABORATORY AND FIELD PERFORMANCE STUDY J. Ryell, B. Chojnacki, G. Woda, and Z. D. Koniuszy	43
BRIDGE DECK DETERIORATION PROMOTED BY ALKALI-CARBONATE REACTION: A DOCUMENTED EXAMPLE M. A. Ozol and H. H. Newlon, Jr.	55
SPONSORSHIP OF THIS RECORD	64

FOREWORD

Most concrete serves its intended function satisfactorily for its design life. Occasionally, however, certain combinations of materials, workmanship, and/or environmental conditions arise that hasten concrete deterioration. This seems to be especially true in the varied and often harsh environment of bridges and pavements. Because the final aspect of deterioration is physical, chemical causes, if they occur, are frequently overlooked. Even if chemical reactions are recognized as contributing to the deterioration, their exact nature is usually not completely understood. The alkali-carbonate rock reaction is one that fits this category. There are still differing opinions as to exactly what the expansion mechanism is, but it has been well established that the reaction is potentially detrimental to concrete.

In the early 1960s the problem was recognized by the Highway Research Board as being important. The Committee on Performance of Concrete-Chemical Aspects sponsored a symposium on alkali-carbonate rock reactions at the 43rd Annual Meeting in 1964. Highway Research Record 45, generated by the symposium and published the same year, contained 15 reports and a related bibliography. In 1969, at the 48th Annual Meeting, the committee again sponsored a conference session, this time called "Alkali-Carbonate Rock Reactions Revisited." The presentations were all informal, with no prepared manuscripts available.

A task group of committee members was appointed in 1970 to see if other unreported data were available that should be brought to the committee's attention. All individuals or organizations known to be studying the alkali-carbonate rock reaction were contacted. At the 1971 committee meeting the task group chairman, Peter Smith, stated that he had the shortest report on record: one word, "nothing." It was found that other researchers had published elsewhere or were prepared to publish their results and did not wish to delay publication. The task group was therefore disbanded.

The speakers making presentations at the 1969 session were, however, asked to prepare manuscripts for publication. This process was begun in 1971. The informal nature of the conference session led to difficulties in obtaining in printed form the material originally presented. Some of those who made presentations had already chosen to publish their work elsewhere. Therefore, related work that has since been submitted by researchers active in the field of study of alkali-carbonate rock reactions is included here.

This RECORD should be of value to geologists, architects, concrete technologists, and materials engineers who specify aggregate and cement for use in concrete. The volume of reactive aggregate can be kept at tolerable levels in a concrete mix, and, when used with low-alkali cement, the expansion can be reduced to acceptable levels for many otherwise unacceptable aggregate sources. In some instances selective quarrying to eliminate reactive layers is called for because this is more effective than the dilution technique.

The papers by Ozol and Newlon, Smith, and Ryell et al. describe field concrete that has been beset by the alkali-carbonate rock reactions. The Ozol and Newlon case study reveals the problems of identifying the contributing causes of deterioration when more than one failure mode is operating.

Smith and Ryell et al. were forewarned "in the middle of the stream" and used selective quarrying, based on laboratory expansion results, to obtain ledges that were not alkali-susceptible. Smith concludes that this was a successful and economical countermeasure. He mentions an accelerated test method that fortifies the normal-alkali "job" cement to 3 percent alkali. Concrete with this enhanced alkali content and reactive carbonate rock gives a clear indication of unacceptable expansion by 21 days. This accelerated test seems worthy of more widespread trials. Accelerated testing of "job" aggregates and "job" cement before they are used could prevent

future problems, although it may create some immediate problems with producers and contractors.

Ryell et al. give a case history of the aggregate from a particular quarry. They have made field performance studies as well as long-term laboratory measurements for individual layers within the quarry. This allowed them to identify the "problem ledges" of rock. Selective quarrying to eliminate the reactive stone was judged more effective than dilution for reducing reaction.

Bukovatz et al. describe D-cracking in Kansas pavements and the research conducted to isolate its cause. The authors report that Kansas limestone coarse aggregate is one of the suspected agents, and they tell of efforts to limit its effect in future construction.

Hilton presents equations to estimate the magnitude of carbonate rock expansion (ASTM C 586-66T) necessary to cause a certain degree of expansion in concrete containing particular volumes of the rock. The necessary dilution of an expansive aggregate with an inert material to keep concrete expansions within certain limits can then be estimated.

Walker has delved into the mineralogy of the reaction and lists some of the reaction products identified in the rock prisms under study. She found brucite in the interior of the reacted aggregate prisms and minerals of the hydrotalcite-sjogrenite groups at the exterior. These minerals were not present in untreated samples. The percent of expansion could not be directly related to the presence or abundance of the minerals found.

Mather describes the history and steps that led to the establishment of the ASTM standard for the rock prism test and the use of the mortar bar test for reactive carbonate rocks. He also discusses the revisions made in Appendix II of the "Standard Practice for Concrete" portion of the Engineer Manual of the Corps of Engineers covering evaluation of reactive aggregates.

—Carl F. Crumpton

KANSAS CONCRETE PAVEMENT PERFORMANCE AS RELATED TO D-CRACKING

John E. Bukovatz, Carl F. Crumpton, and Herbert E. Worley,
State Highway Commission of Kansas

D-crack deterioration in portland cement concrete pavements was common in Kansas in the 1930s; however, no large-scale investigation was made until 1944. That survey indicated a significant relationship between coarse aggregate and D-cracking. The aggregate size was reduced and aggregate testing was improved. Another pavement condition survey was made in 1951-1952 to determine the effect of materials or designs on deterioration. Again coarse aggregate came under suspicion. For a time after this survey it appeared that there was little additional trouble with D-cracking. In 1962 deterioration was noticed in pavements less than 5 years old, and in 1964 another study was begun with objectives of determining the extent of damage, reasons for its occurrence, and effective methods of repairing existing damage and preventing future damage. This report covers pavement performance as related to the extent of damage and clues for the prevention of future damage. The following conclusions were reached: D-cracking is still a problem in Kansas. All Kansas limestones used in pavement concrete have been associated with D-cracking. Pavements with limestone coarse aggregates in excess of 35 percent were more likely to be D-cracked than pavements with less than 35 percent limestone coarse aggregates. Most of the pavements without limestone coarse aggregate were rated good except those constructed with Blue River, Walnut River, or Bazaar gravel deposits. Implementation of these results has included limiting the limestone coarse aggregate to no more than 30 percent, with a maximum size of $\frac{1}{2}$ in., and using type II cement and preformed neoprene transverse joint sealers.

•DETERIORATION of concrete pavements was common in Kansas in the 1930s. The terms D-line or D-crack are used to describe the disintegration, which is characterized by many fine, closely spaced, parallel cracks, usually with a black, blue, gray, or white deposit in the portion of the crack at the pavement surface. The types of deterioration referred to have been defined previously, and usage in this report conforms to those definitions (3, 4, 6, 14, 16, 18, 19, 21).

Prior to 1944 no large-scale investigation of Kansas concrete pavements had been made, but at that time a survey was made of 1,170 miles of bare concrete pavements spanning a construction period of 27 years (1919-1945). The results of the survey were reported by White and Peyton (22). The purpose of the survey was to investigate the relationship between certain classes of failure (mainly D-lines) and the materials incorporated into the concrete (principally the coarse aggregate). White and Peyton reported a distinct relationship between the performance of the pavements, as related to D-crack damage, and the source of the coarse aggregate. The limestone aggregates from the sources listed in their report, which were most often associated with poor pavement ratings, are the Argentine, Ervine Creek, Plattsmouth, and Stoner limestones (all of Pennsylvanian age). Some nonlimestone aggregates—Bazaar, Walnut

River, and Blue River gravels—were also associated with poor pavement ratings. White and Peyton recognized that the cause and effect of such failures may not have been limited to the relationship between the coarse aggregate and D-crack failures. They mainly wanted to convey the thought that this particular relationship was highly significant.

As a result of that study (22) and related investigations conducted at the same time, certain changes were made in coarse aggregate specifications. The maximum size of aggregates from some sources was reduced. Changes were made in the freeze-thaw test procedures to make the test more effective in evaluating the quality of an aggregate and in predicting its performance in portland cement concrete.

Another survey similar to the one made by White and Peyton was conducted in 1951 and 1952 by the State Highway Commission of Kansas and the Portland Cement Association. This survey included 1,165 miles of in-service bare concrete pavement constructed between 1921 and 1949 and 248 miles of covered concrete pavement constructed between 1919 and 1937. The purpose of the survey was to determine if materials or designs influenced the existence of or lack of pavement deterioration. The survey found that the older pavements had a much poorer rating than the newer pavements, except for newer pavements constructed with a low cement factor (1.25 bbl/yd^3), which also had poor ratings. The results of the 1951-1952 study indicated that the Ervine Creek, Plattsmouth, and Stoner limestones and the Blue River sand-gravel were associated with unsatisfactory pavement performance. However, because of the many variables involved in this study, it was usually not possible to pinpoint the cause for the deterioration found in the field.

For a period of time after the 1951-1952 survey it appeared that there was little additional trouble with D-cracking. In 1962, however, several new pavements were observed to have a stain pattern adjacent to joints or at uncontrolled cracks. This stained area often progressed into D-cracking, with subsequent raveling of the pavement near joints or cracks. By 1963 the occurrence of this type of deterioration was noted in some pavements that were less than 5 years old.

In 1964 a portland cement concrete pavement deterioration study was begun with the objective of determining the extent of the damage, the reasons for its occurrence, the most effective method for the repair of already damaged pavement, and changes in procedures or materials required to prevent or reduce such deterioration on future work. Kansas published an initial report (2) covering the field operations specifically related to the extent of the damage and clues for the prevention of future damage. Additional reports will cover other phases of the study.

PROCEDURES

A spot check of several projects was made in the field prior to a full-scale field survey as a planning aid and to check worksheets and procedures. The final field survey consisted of a walking examination of one-half of each of the 1,200 two-lane miles of bare concrete pavements and recording various pavement defects. Figure 1 shows D-cracks at a transverse crack, and Figure 2 shows stain and D-cracks at transverse joints.

Other field work consisted of coring some of the pavements surveyed. The coring sites were selected to obtain a sampling that represented the variables of coarse aggregate, joint construction, and amount of deterioration. Each section chosen was then cored in the best and worst areas. The cores were taken to the laboratory for testing. Microscopic examination and tests to determine compressive and tensile strength, original water content, void system parameters, and freeze-thaw durability were performed, but these studies are not covered in this report.

SURVEY RESULTS

Pavements were rated good, fair, or poor. By this system, if the percent of pavement panels affected by D-cracking ranged from 0 to 12 percent, the pavement was considered good. When the percentage was from 13 to 50 percent it was considered fair, and if more than 50 percent of the panels were D-cracked or raveled the rating

Figure 1. D-cracks at an uncontrolled transverse crack of a concrete roadway.

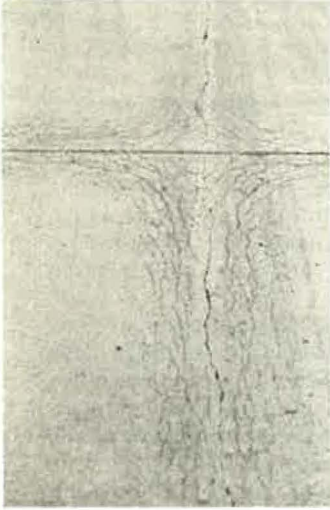


Figure 2. Stained and D-cracked transverse sawed joints.



Table 1. Comparison of surveys.

Rating	Percentage of Pavement Surveyed		
	1944-1945	1951-1952	1964-1965
Good	54	65	74
Fair	8	35	8
Poor	38		18

Table 2. Miles rated for various percentages of limestone in the concrete, 1964-1965 survey.

Rating	Miles for Various Percentages of Limestone		
	None	1-35 Percent	36-70 Percent
Good	250	327	182
Fair	25	9	43
Poor	16	8	154

Table 3. Two-lane miles of good, fair, and poor ratings for age versus percentage of limestone.

Age (years)	None			1-35 Percent			36-70 Percent		
	Good	Fair	Poor	Good	Fair	Poor	Good	Fair	Poor
0-5	15	7	6	211	9	8	64	3	23
6-10	9	3	0	54	0	0	62	22	38
11-15	52	2	4	52	0	0	23	11	38
16-20	51	6	0	10	0	0	10	0	8
21-25	29	0	1	0	0	0	0	2	8
26-30	45	3	0	0	0	0	13	1	3
>30	49	4	5	0	0	0	10	4	36

was poor. The good, fair, and poor rating system was used by White and Peyton (22). The data of the 1951-1952 survey were arranged in this manner, except that only the pavements rated good or otherwise were differentiated. The similarity in the rating system of the three surveys and the fact that each survey covered approximately 1,200 two-lane miles of bare concrete pavement provide a basis for comparison between surveys. Table 1 gives the ratings of the surveys. The 1964-1965 survey showed that D-cracking was still a problem, even though there was a 9 percent increase in the amount of pavement rated good since the 1951-1952 survey. The early appearance of D-cracking in new pavements and 18 percent of the pavements rated as poor (Table 1) were evidence that the problem still existed. During the period from the 1951-1952 survey to the 1964-1965 survey the policy of keeping pavements free from ice and snow by using deicing salts may, in part, account for the early signs of D-cracking. During this same period it has been found that bridge deck damage also increased (6, 7, 8, 9, 10, 11, 17). We, like others, believe that salt acts to accelerate deterioration of concrete that lacks frost resistance.

Limestone coarse aggregate used in concrete has been under suspicion since the early surveys. Every limestone geologic unit used in Kansas concrete highway pavements has been associated with D-cracking at one time or another somewhere in the state. Even though no Kansas limestone coarse aggregate has a blemish-free record, most limestone coarse aggregates are associated with long or short stretches of concrete pavement that have little or no D-cracking. Some of these pavements are 30 to 40 years old and still in good condition. Stingley and Worley (20) found that concrete bridge decks that performed both well and poorly have been constructed in Kansas using these same aggregates.

To place the observations in proper perspective, the percentage of limestone used in the various concrete pavements was summarized for each of the pavement ratings of good, fair, and poor. The data are given in Table 2. The two-lane miles listed include most but not all of the pavements surveyed because of the lack of information in the files regarding percentages of coarse aggregates for a few of the older projects. This table shows that the use of 35 percent or less of Kansas limestone in concrete pavement gives a high probability of good performance. This is in contrast to the use of more than 35 percent Kansas limestone, which has about as great a probability of being poor as good. The limestone aggregates most often associated with pavements rated poor were the Argentine, Bethany Falls, Ervine Creek, Plattsmouth, Stoner, and Winterset formations, all of the Pennsylvanian age, and the Cresswell formation, of Permian age. It should be remembered, however, that all Kansas limestones used in concrete pavements have produced D-cracking somewhere in the state. Most of the pavements made without limestone (sand-gravel, sandstone, or chat-sized chert, which is a waste product of lead and zinc mining operations) are in the good category. The nonlimestone aggregates that are in the fair and poor ranges are mostly Blue River sand-gravel, with a few miles having Walnut River or Bazaar gravels. Most of the pavements using Blue River sand-gravel that were rated poor were made with aggregate having a top size of $1\frac{1}{2}$ in. The Blue River sand-gravel has a good record for not producing map cracking (13), but all three surveys have implicated it as a D-cracking aggregate.

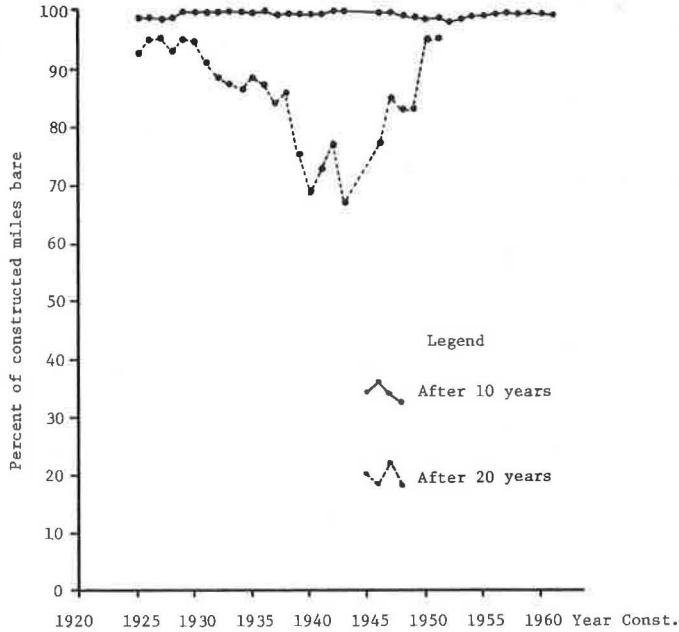
Table 2 does not take into account the age factor; therefore, Table 3 was constructed to relate the good, fair, and poor ratings to both age and percentage of limestone. Table 3 also shows that, for comparable ages, if the concrete contains a lower percentage of limestone or no limestone, there is a much higher ratio of good to fair and poor.

Since the 1964-1965 survey, data relative to the pavement age when overlaid have been assembled for the 51-year period from 1921 to 1972. Some of the data are summarized in Table 4. This table gives the percentage of miles still bare in 1972 for each construction year since 1921. The percentages of the pavements still bare at ages of 10 and 20 years were plotted separately as 5-year moving averages in Figure 3. These plots show that we are now in a betterment trend in pavement life after a few bad years that caused a downward trend. The bad years (1939, 1940, 1943) that caused the dips in the moving average lines are actually not as disastrous as it seems,

Table 4. Miles of concrete pavement constructed each year and still bare in 1972.

Construction Year	Age to 1972	Miles Constructed	Percent Bare, May 1972
1921	51	27.9	0
1922	50	91.1	0
1923	49	94.3	6.3
1924	48	46.9	5.5
1925	47	40.1	15.9
1926	46	23.8	17.2
1927	45	94.5	15.6
1928	44	86.1	6.7
1929	43	38.9	46.1
1930	42	57.5	21.1
1931	41	131.5	16.1
1932	40	64.0	42.0
1933	39	33.6	18.9
1934	38	49.5	42.3
1935	37	70.9	1.2
1936	36	77.5	16.0
1937	35	70.8	14.7
1938	34	26.4	26.6
1939	33	27.4	8.1
1940	32	17.6	22.2
1941	31	25.1	66.9
1942	30	17.6	88.6
1943	29	2.6	0
1946	26	27.9	100.0
1947	25	39.2	72.9
1948	24	45.8	85.1
1949	23	70.1	95.4
1950	22	49.2	97.0
1951	21	14.3	100.0
1952	20	43.3	97.7
1953	19	57.4	87.1
1954	18	35.3	91.8
1955	17	52.7	84.4
1956	16	51.0	100.0
1957	15	26.7	100.0
1958	14	94.4	94.6
1959	13	108.9	99.4
1960	12	94.1	100.0
1961	11	92.6	98.2
1962	10	51.4	100.0
1963	9	38.3	100.0
1964	8	64.4	100.0
1965	7	43.5	100.0
1966	6	31.5	100.0
1967	5	54.2	100.0
1968	4	31.0	100.0
1969	3	65.0	100.0
1970	2	59.3	100.0
1971	1	106.7	100.0

Figure 3. Five-year moving averages of pavements bare after 10 and 20 years for each year of construction.



since the total miles constructed in these years were small. The improving trend is presumably a reflection of implementation of changes made as a result of the 1944-1945 and 1951-1952 D-crack pavement studies.

OTHER OBSERVATIONS

Subgrade paper was used under some of the older pavements. Cores taken from some of these pavements (usually in good condition) still had the subgrade paper adhering to the bottom. The side of the paper next to the subgrade had developed a clay slickenside, which indicated that the pavement slab had been sliding as a unit on the clay subgrade. If this is so, it may have resulted in decreasing tensile stress in the slab, with the effect of reducing cracking.

Cores taken from pavements that contain steel mesh often have voids adjacent to the transverse wires of the mesh. In many cases the void parallel to the mesh wire continued completely through the core. This type of void was also observed recently in a core taken from a bridge constructed in 1914 that used mesh wire in the top part of the deck.

Sometimes during the slip-form paving operation a section at the vertical pavement edge would fall, or break away, while the concrete was still plastic after the paver had passed. The outline left by the outfall was a flattened cone, with the sloping sides trending to the plane of the mesh. A considerable amount of water would then flow from next to the end of a mesh wire, much like a miniature spring. It appeared that there was a water gain area at the level of the mesh and that hydraulic pressure was the cause for the plastic concrete outfall. This makes us wonder if the mesh in pavements or the reinforcing steel in bridge decks sets up an electric field in plastic concrete that attracts and temporarily holds the water at the plane of steel where water gain is common (1, 5). We have seen elongated water voids close to the steel such that the voids line up in patterns that remind us of electric current or magnetic flow lines. It is speculative, of course, but this may be a case of natural electro-osmosis occurring where the water migrates from the paste to the steel. This is envisioned to be somewhat similar to the thermo-osmotic and thermo-electric coupling in saturated soils described by Gray (12) but probably not due to temperature differences.

Studies have been made in our laboratory of electric currents within reinforced concrete and the effect of the system pH on current flow. These studies indicate that electrical potentials exist within reinforced concrete and that the pH of the system does influence the electrical activity. The exact significance of the findings is not certain; however, in both studies water in some form was necessary. We presently have studies under way utilizing the principles of electro-endosmosis to move polar liquids and dissolved substances into (or out of) hardened concrete in an attempt to improve the durability of the concrete. The technique looks quite promising because several substances appear to move in hardened concrete under an electric current.

The effect of water in either plastic or hardened concrete should not be ignored. It has been observed that excessive mixing water has produced poor-quality concrete. Kansas concrete pavements deteriorate at the joints or cracks where the rain or evaporating subsurface water is most likely to be concentrated. Tests in our laboratory indicate that the drier a concrete is, the more durable it is in a freeze-thaw environment. While water is not the only factor in concrete pavement deterioration, its presence in some form is a great liability. For example, water is required to form the Ca(OH)_2 found in D-cracked concrete. Water is required for alkali-silica and alkali-carbonate reaction. Water is needed to form the calcium carbonate hexahydrate (15) that we have found in D-cracked concrete. Water is needed for internal expansion related to freezing and to blowups during hot weather. Water carries deicing salts into the concrete, where the salt solution acts to corrode the steel.

We see rust stains on the surface of our pavements, showing that the mesh, tie bars, and load transfer devices are corroding. This is probably an indication of future pavement problems related to the expansive forces of the steel corrosion products. A means of controlling the water in concrete would do much to control deterioration of concrete.

Joints in general are problems in Kansas concrete pavements. Joint or sliver spalls are common during the early years of a pavement's life. The early presence of the spalls is usually forgotten after a joint has D-cracked. The original spall is then often considered as raveling due to D-cracking, which may not be the case. Joint or sliver spalls along with D-cracking were a common type of defect listed on the tally sheets of both the 1964-1965 and 1951-1952 surveys. It should be kept in mind that even poor pavements are poor only at joints or cracks and that most of the panel is still in good shape, but rideability may be poor because of the conditions. Improvements in jointing techniques, joint sealing practices, joint spacing, and perhaps joint widths would seem to be a desirable goal to help improve concrete pavement longevity in Kansas. We do not believe that continuously reinforced pavement is the answer, either, because with our aggregates we could then have D-cracking at a very much larger number of transverse cracks.

IMPLEMENTATION

During the period from 1964 to 1971, changes have been made in pavement concreting practices (materials, methods, and maintenance). Some of the changes were the direct result of this study and others were due to some general observations that were strengthened by what other states were doing. These changes are as follows:

1. Subgrade—Lime treatment is required in most locations.
2. Methods—Vibrators are required; slip-form pavers are allowed and encouraged; mesh depressers are used rather than two-course paving.
3. Materials—Type II cement is required; use of limestone aggregate is limited to no more than 30 percent unless subgrade paper and parting plates are used, in which case the maximum limestone allowed is 50 percent; maximum size of limestone coarse aggregate has been reduced to $\frac{1}{2}$ in.
4. Joints—Preformed neoprene transverse joint sealers are used.
5. Repairs—Some D-cracked joints and uncontrolled cracks have been repaired by removing the deteriorated concrete and replacing it with either portland cement concrete or a bituminous mix; some of the pavements rated as poor have been overlaid with bituminous concrete both with and without previous repair of D-cracked areas.

CONCLUSIONS

1. D-cracking is still a problem in Kansas.
2. All Kansas limestones used in pavement concrete have been associated with D-cracking.
3. Pavements with limestone coarse aggregate in excess of 35 percent were more likely to be D-cracked than pavements with less than 35 percent limestone coarse aggregates.
4. Most of the pavements without limestone coarse aggregates were rated as good except those constructed with Blue River, Walnut River, or Bazaar gravel deposits.
5. Results of the studies can be used to modify former practices.

REFERENCES

1. Bukovatz, John E. Bridge Deck Deterioration Study, Part 7: Weathering Test of Reinforced Concrete Slab With Various Depths of Steel. State Highway Commission of Kansas and U.S. Bureau of Public Roads, 1968.
2. Bukovatz, J. E., Crumpton, C. F., and Worley, H. E. Study of D-Cracking in Portland Cement Concrete Pavements, Report 1: Field Phase. State Highway Commission of Kansas and Federal Highway Administration, 1973.
3. Cement and Concrete Terminology. American Concrete Institute, publication SP-19, 1967.
4. Cordon, William A. Freezing and Thawing of Concrete: Mechanisms and Control. American Concrete Institute Monograph No. 3, 1966.
5. Crumpton, Carl F., Pattengill, Maurice G., and Badgley, William A. Bridge Deck Deterioration Study, Part 8: Special Study of Blue Rapids Bridge Deck. State Highway Commission of Kansas and Federal Highway Administration, 1969.

6. Durability of Concrete Bridge Decks: A Cooperative Study. State Highway Commission of Kansas, U.S. Bureau of Public Roads, and Portland Cement Association, Rept. 1, 1965.
7. Durability of Concrete Bridge Decks: A Cooperative Study. Michigan State Highway Department, U.S. Bureau of Public Roads, and Portland Cement Association, Rept. 2, 1965.
8. Durability of Concrete Bridge Decks: A Cooperative Study. California Division of Highways, U.S. Bureau of Public Roads, and Portland Cement Association, Rept. 3, 1967.
9. Durability of Concrete Bridge Decks: A Cooperative Study. Missouri State Highway Commission, U.S. Bureau of Public Roads, and Portland Cement Association, Rept. 4, 1968.
10. Durability of Concrete Bridge Decks: A Cooperative Study. California, Illinois, Michigan, Minnesota, New Jersey, Ohio, Texas, and Virginia highway departments, U.S. Bureau of Public Roads, and Portland Cement Association, Rept. 5, 1969.
11. Durability of Concrete Bridge Decks: A Cooperative Study. California, Illinois, Kansas, Michigan, Minnesota, Missouri, New Jersey, Ohio, Texas, Virginia highway departments, U.S. Bureau of Public Roads, and Portland Cement Association, Final Report, 1970.
12. Gray, Donald H. Thermo-Osmotic and Thermoelectric Coupling in Saturated Soils. HRB Spec. Rept. 103, 1969, pp. 66-77.
13. Hadley, David W. Field and Laboratory Studies on the Reactivity of Sand-Gravel Aggregates. Portland Cement Association, Jour. of the Research and Development Laboratories, Vol. 1, No. 3, Sept. 1959, pp. 17-33.
14. McElherne, T. E., Welp, T. L., Roberts, S. E., and Ellingson, W. A. Iowa "D"-Crack Problem. Iowa State Highway Commission, 1967.
15. Pattengill, M. G., Crumpton, C. F., and Badgley, W. A. An Occurrence of a Hydrated Form of Calcium Carbonate in Deteriorated Concrete. Bull. of the Association of Engineering Geologists, Vol. 9, No. 1, Winter 1972, pp. 47-56.
16. Pavement Condition Surveys: Suggested Criteria. HRB Spec. Rept. 30, 1957.
17. Reagel, F. V. Introduction, Effects of De-Icing Chemicals on Structures: A Symposium. HRB Bull. 323, 1962, pp. 1-2.
18. Standard Nomenclature and Definitions for Pavement Components and Deficiencies. HRB Spec. Rept. 113, 1970.
19. Stark, David. Discussion of "Laboratory Freeze-Thaw Tests vs Outdoor Exposure Tests" by Axon, E. O., Murray, L. T., and Rucker, R. M. Highway Research Record 268, 1969, pp. 45-46.
20. Stingley, W. M., and Worley, H. E. Bridge Deck Deterioration Study, Part 1: A Comparison of 777 Uncovered Decks. State Highway Commission of Kansas and Federal Highway Administration, 1971.
21. Welp, T. L., and McElherne, T. E. Iowa Field Conference on D-Cracking. Iowa State Highway Commission, 1967.
22. White, L. V., and Peyton, R. L. Condition of Concrete Pavements in Kansas as Affected by Coarse Aggregate. HRB Proc., Vol. 25, 1945, pp. 129-146.

EXPANSION OF REACTIVE CARBONATE ROCKS UNDER RESTRAINT

M. H. Hilton, Virginia Highway and Transportation Research Council, Charlottesville

It is suggested that the degree of expansion of alkali-reactive carbonate rocks is not only a function of the volume of dolomite contained in the rock but also of the texture of the material as reflected by its deformation characteristics under sustained loading. Theoretical equations are presented that can be used for calculating the expansion of a two-phase system composed of certain volumes of expanding elastic particles enclosed and restrained within an elastic matrix. A comparison of calculated expansions with the expansions of concrete measured experimentally indicates that the theoretical relationship gives a good approximation of the expansion of a two-phase system such as concrete. The solution is then applied to the analogous carbonate rock case, which is considered as a two-phase system composed of dolomite particles enclosed within a calcite-clay matrix. Carbonate rock samples, which were chosen with regard to their expansive reactivity in alkaline solution, were analyzed physically, chemically, and petrographically. The experimental data thus obtained, supported by the theoretical analysis, indicate that the degree of expansion of a reactive carbonate rock depends on a complex interplay of its structural texture and compositional properties. The deformation characteristics of a carbonate rock under sustained loading tend to reflect its structural rigidity, which in turn is a significant property when the effects of either textural or external restraints to expansion are considered. Thus a reactive carbonate rock with low textural restraint could be highly expansive when unrestrained, but when restrained in a mass of concrete it would be more compressible and less expansive.

•FROM the results of various tests, Hadley (1) postulated a dedolomitization reaction (the replacement of dolomite by calcite and brucite) between dolomite and alkali metal hydroxides as the basic reaction mechanism leading to the expansion of certain carbonate rocks. This reaction mechanism theory has been generally supported by the findings of others (2, 3, 4, 5). While several hypotheses have been proposed to explain the mechanism of expansion (6, 7, 8, 9, 30), all are based on expansion originating from the dolomite crystals following an initial dedolomitization reaction. As research has progressed, however, the classification of potentially expansive carbonate rocks has broadened considerably. The results of earlier studies by Hadley and others indicated that the majority of the expansive rocks could be classified as fine-grained, argillaceous, and dolomitic (1, 3, 10, 11). The rock texture was found to be that of small dolomite crystals dispersed in a matrix of microcrystalline calcite and clay material. The carbonate portion of the rock generally contained 40 to 60 percent dolomite, and insoluble residues consisting mostly of clays and other impurities constituted a 10 to 20 percent portion of the material. Other authors (12, 13) have reported on some expansive rocks that did not fit these criteria, and in some additional studies Hadley (14) found that carbonate rocks containing from 80 to 100 percent dolomite could be highly expansive when reacted in alkaline solutions. Hadley further suggested that clay content

was an important factor, since interstitial clays could weaken the carbonate skeleton and make a rock more susceptible to expansive pressures. In addition, Dolar-Mantuani (15) has reported on some carbonate rocks that did not expand until immersed in a sodium hydroxide solution for at least 30 weeks, whereas the more typically expansive rocks show significant expansions after only a few days of exposure. Thus, one might conclude from a review of the prior research that the potential for carbonate rock expansion and the rate of expansion will depend on a complex interplay of the rock's composition, the size and volume of its various components, and the structural rigidity of the total material.

CARBONATE ROCK STRUCTURE

In addition to the work mentioned previously, Hadley (1) conducted tests on a comparatively large 0.3-in. (7.6-mm) gauge length dolomite crystal and observed an expansion of 0.15 percent after 100 days' immersion in a sodium hydroxide solution. While the degree of expansion was small when compared to that of many reactive rocks, the dolomite crystal was considerably larger than the order of 30-micron size found in most expansive rocks. In view of other experimental evidence showing that the rate of dolomite reaction increases with decreasing particle size (1), the size difference between the experimental crystal and those found in expansive rocks would appear to be significant. Thus, the migration of alkalis into the structure of a large dolomite crystal would probably require considerably more time than that required to penetrate an equivalent volume of dispersed microscopic dolomite rhombs. Accordingly, one might expect the more porous clay media of the readily expansive rocks to provide access channels to the small dolomite rhombs. Expansion might then proceed at a high rate when the total material has a weak structure and contains a sufficient volume of dispersed microscopic dolomite particles. On the other hand, rocks with high carbonate contents would tend to have strong interlocking of the carbonate crystals, and hence the alkalis could not readily penetrate the structure, which would, in turn, tend to be strong and rigid.

The approximate volume of carbonate particles at which the interlocking of the crystalline structure of a rock might be expected is indicated from the work of Hsu (16), who determined that, in the close-packed state, a face-centered cubic lattice of spherical particles would occupy 74.2 percent of the total cubic volume. Although carbonate particles are not normally spherically shaped, this relationship does suggest a volume in which crystal interlocking might be expected. Furthermore, from a mechanics viewpoint, one would expect rocks with interlocking of the carbonate constituents to be comparatively rigid and have higher moduli of elasticity than rocks with little interlocking of the carbonates. It would follow that rocks with higher moduli of elasticity would have more textural restraint due to a more rigid structure and thus less tendency to expand.

The typical structure of the more expansive carbonate rocks is quite similar to that of portland cement concrete. While concrete can be considered as a two-phase system consisting of aggregate particles dispersed in a cement-mortar matrix, the expansive-type rocks consist of dolomite rhombs dispersed in a matrix of clay, calcite, and miscellaneous impurities. Thus, the ultimate expansion of either system would depend on the volume of expansive particles present, their degree of reactivity, and the textural rigidity of each phase. Furthermore, if expansion is affected by the rigidity of a reactive rock's structural fabric, then the expansion of concrete containing the material could not be expected to be proportional to the expansion of rock samples reacted in alkaline solutions.

PURPOSE AND SCOPE

Reactive carbonate aggregates are usually detected in the laboratory by measuring the expansion of small rock specimens reacted in alkaline solutions by procedures similar to those in ASTM C 586 or by periodically measuring the expansion of concrete specimens containing the material. Accordingly, two types of restraint can be considered as countering the expansion of reactive carbonate rocks. These are the restraints that would be imposed by the texture (or structural fabric) of the rock and those

that would be imposed externally. In the latter case, the typical restraint would be that of the cement-mortar matrix surrounding the aggregate particles in concrete. The purpose of the study was to investigate the effects of each of these two types of restraint.

The scope of the work was limited to an investigation of the expansion and textural characteristics of some carbonate rock samples selected with regard to their magnitude of expansion when reacted in a 1N NaOH solution.

THEORETICAL CONSIDERATIONS

Although several expansion mechanisms have been proposed, as referenced earlier, each assumes expansion to be centered around the dolomite phase of the rock. Accordingly, a carbonate rock can be considered as a two-phase system composed of dolomite expanding within a surrounding matrix.

Elongation, or expansion, of a particular body of material can be defined in terms of stresses, moduli of elasticity, and geometrics. If a spherical body composed of a volume of one material surrounded by the matrix of another is assumed, the relationship between the expansion of the core volume and the expansion of the total volume can be developed from equations derived by Lamé (17) that define the stresses in the body. In addition, by using the relationships between stresses and strains and by taking Poisson's ratio as 0.2, which is a reasonable value for masonry and rock materials (18), an equation for the expansion of the body, ξ_b , was developed earlier (19) that takes the form

$$\xi_b = \frac{\xi_s}{\frac{1-g}{3g} \left(\frac{1.5E_m + 0.5}{E_s} \right) + \frac{1+2g}{3g}} \quad (1)$$

This relationship indicates that the unit linear expansion of the total body, ξ_b , varies directly as the unit linear expansion of the core material, ξ_s , and inversely as the fractional volume of the core, g , and the ratio of the modulus of elasticity of the matrix, E_m , to that of the core, E_s .

The restrained unit linear expansion of the core material, $\xi_{s(r)}$, has also been developed (19); it takes the form

$$\xi_{s(r)} = \xi_s - 1.5 \xi_b \frac{E_m}{E_s} \left(\frac{1-g}{3g} \right) \quad (2)$$

Experimental Versus Calculated Expansions

Newlon and Sherwood (20) have presented experimental data on the 1-year expansions of moist-cured concrete prisms made with an expansive aggregate (sample 1-8). These data show the effects on concrete expansion of cement alkali content and dilution of the expansive aggregate volume with a reference granite material. Since concrete can be considered as a two-phase system, these data offer a convenient means of comparing calculated and experimental expansions.

The relative volumes of aggregate and cement mortar in the hardened concrete prisms can be calculated from the original mix proportions by use of a relationship developed by Axon (18). Typical values for the modulus of elasticity of cement mortar range from 1 to 4 million psi and depend on the water-cement ratio and age (21). Because the elasticity of the mortar phase would vary with time, it would appear reasonable to assume an average effective E_m of 2.5×10^6 psi (17.25×10^6 kPa). Because of the long-term stresses induced in the materials by the expanding aggregate, considerable creep would also be involved. To account for this phenomenon the elastic modulus of cement mortar is normally reduced to approximately one-third of the actual value (22). A similar elasticity value (given in Table 3) must also be used for the aggregate sample 1-8. Accordingly, an effective E_m/E_s ratio of 0.48 was obtained that takes the time-dependent creep effect into account. Thus, for a given fractional volume of expansive aggregate, the unrestrained unit expansions of the aggregate, ξ_s , were calcu-

lated by use of Eq. 1. These values established the 1-year unit expansions of the aggregate reaction with a given cement alkali content. The concrete expansions, ξ_s , for each remaining value of the fractional volume of aggregate, g , were calculated and plotted against the experimental data (Figure 1). A very good correlation between the experimental and calculated expansions was obtained.

Equation 1 can be used to estimate concrete expansions that may result from reactive rock expansions measured by test methods such as ASTM C 586. It should be noted, however, that cracking can develop in concrete at expansions on the order of 0.05 percent (10, 11). Although minor isolated cracking should not have much effect on calculated estimates of expansion, the theory does not consider cracking in the matrix or in the expanding particles. In addition, the upper limit of g in Eqs. 1 and 2 would be on the order of 0.74 (for particles approaching spherical shapes, for example), because at this volume the matrix and particle relationship would reverse; i.e., the matrix would become enclosed in the surrounding aggregate material. Finally, effective E_m and E_g values cannot be determined from standardized tests and must be estimated either by applying approximate corrections to normal E values or by estimating the effective values by testing the materials under sustained loading.

MATERIALS, TESTS, AND PROCEDURES

Rocks Studied

A variety of carbonate rock samples that exhibited different magnitudes of expansion when reacted in a 1N NaOH solution were selected for study. The rock sample code numbers, the general source of the materials, and references to additional studies involving the material are given in Table 1.

Cylindrical specimens 0.45 in. (1.14 cm) in diameter and 1 in. (2.54 cm) and $1\frac{3}{8}$ in. (3.5 cm) long were drilled from the rock samples so that their axes were perpendicular to the sedimentation layers. The longer cylinders were tapered 45 deg on each end and used for unrestrained expansion measurements. The shorter cylinders were used for restrained expansion measurements and other physical tests. Material adjacent to that from which the cylinders were taken was used for chemical and petrographic analyses and for porosity determinations.

Composition and Texture

The major constituents of the rock samples were determined by the chemical method described by Bisque (25) and are given as percent by volume in Table 2. Although some petrographic data are given in Table 2, this information was used primarily to determine the relative textural characteristics of the rock samples.

The general texture of rocks 1-6, 1-8, 12-9B, 27-4, and K-B was similar to that of 12-9A shown in Figure 2, which clearly shows the similarity between the texture of some carbonate rocks and that of portland cement concrete in section. The major difference between the textures of this group of rocks was the relative amounts of the various constituents and the intergranular contact between the dolomite rhombs. In sample 1-6, for example, the dolomite rhombs were fewer and more widely dispersed in a matrix phase of calcite, quartz grains, and partially silicified or pyritized fossils. On the other hand, sample 12-9B contained a larger volume of dolomite with more contact between the individual grains than did 12-9A. Rock K-B was similar to the latter material except that in some areas the dolomite rhombs were not well sorted and were partially locked in by sparry calcite. In this respect rock 12-U was similar to K-B. The texture of sample 1-8 was similar to that shown in Figure 2 except that the dolomite was not as sharply defined and the matrix contained more clay-like material.

A photomicrograph of the most expansive sample, Mo, is shown in Figure 3. The matrix of this rock was the most porous of all the study materials, and it occupied smaller areas between the relatively high volume of discrete dolomite rhombs.

Sample B-A (Figure 4) had a very rigid structure composed almost entirely of dolomite, with some interstitial carbonate and quartz grains present. This rock was selected primarily for use as a reference because of its high dolomite content and be-

Figure 1. Comparison of computed expansions with the expansion of 4 x 4 x 11-in. concrete prisms containing variable volumes of expansive aggregate sample 1-8 (experimental data from 20).

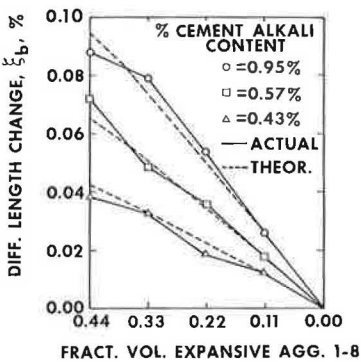


Table 1. Carbonate rocks selected for study.

Sample Designation	General Location of Source of Rock Samples	Additional References
1-6	Northwest Virginia	(10, 23)
1-8	Northwest Virginia	(2, 10, 20, 23)
12-9A	Southwest Virginia	(23)
12-9B	Southwest Virginia	(23)
12-U	Southwest Virginia	(23)
27-4	Central-western Virginia	(23)
B-A	Central-western Virginia	(23)
K-B	Kingston, Ontario, Canada	(24)
Mo ^a	Missouri	(12)

^aThis sample is the same as that designated as sample 8bb in Table 2 of Axon and Lind (12).

Table 2. Compositional and physical data of carbonate rock samples.

Sample Designation	Chemical Analyses ^a			Petrographic Analyses: Average Size of Dolomite Rhombs (microns)	Absorption (%)	Bulk Specific Gravity	Porosity (%)
	Volume Dolomite (%)	Volume Calcite (%)	Volume Insoluble Residue (%)				
1-6	15.8	68.7	14.9	40	0.20	2.76	0.552
1-8	32.4	36.7	29.7	41	0.451	2.78	1.254
12-9A	28.9	55.4	14.5	30	0.415	2.75	1.141
12-9B	44.6	39.3	14.8	34	0.480	2.74	1.315
12-U	27.8	61.9	9.9	40	0.133	2.72	0.362
27-4	19.4	67.9	12.1	35	0.215	2.74	0.589
B-A	94 ^b	— ^b	— ^b	90	0.064	2.84	0.182
K-B	44.3	46.3	8.7	40	0.262	2.79	0.731
Mo	58.6	23.5	12.3	34	2.11	2.67	5.634

^aVolume percentages were calculated from chemical analyses weight percentages by using the following specific gravities: dolomite, 2.85; calcite, 2.71; insoluble residue, 2.70 (26).

^bVolumes determined by petrographic analysis. Calcite, clays, and other impurities make up the remainder of the total volume.

Figure 2. General texture of sample 12-9A showing discrete dolomite rhombs dispersed in a fine-grained matrix of microcrystalline calcite and clay (100 X).

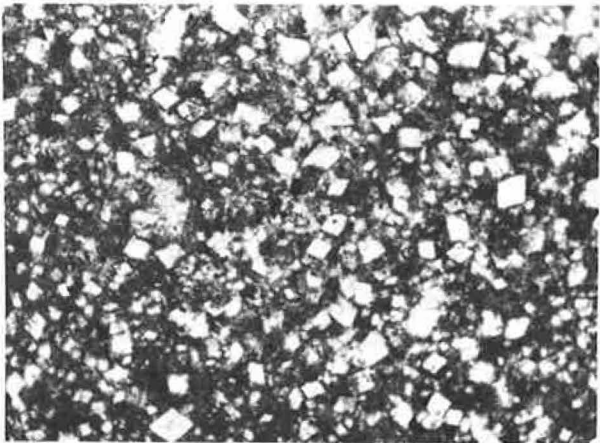


Figure 3. General texture of sample Mo showing a high dolomite content and porous matrix (100 X). Some of the clear areas are due partially to a loss of grains during preparation of the thin section.

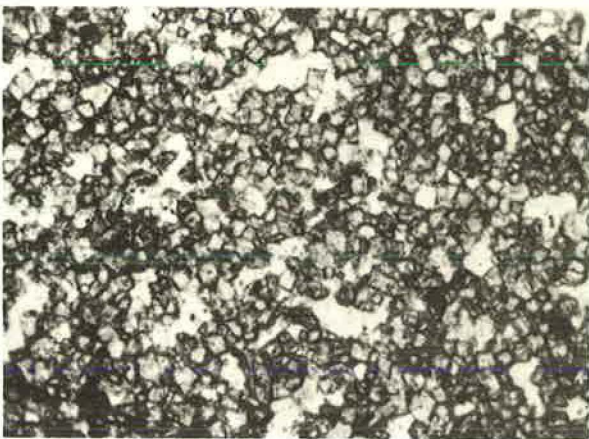


Figure 4. Texture of sample B-A showing a mosaic of equant dolomite grains forming a very homogeneous, virtually nonporous structure (100 X).

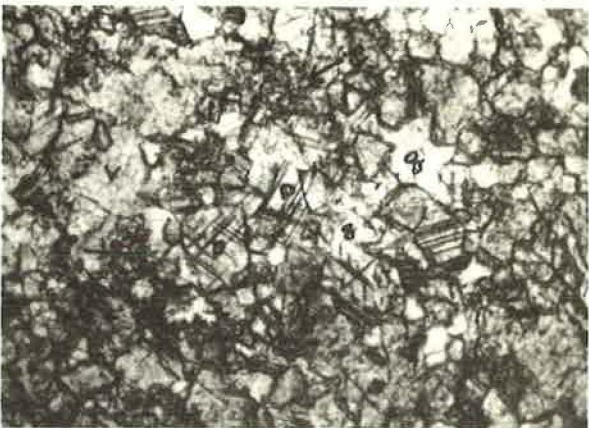


Table 3. Sustained modulus of elasticity of carbonate rock samples.

Sample Designation	Slope, E_s , Sustained Modulus, $\times 10^6$ (psi)	95% Confidence Range for Sustained Modulus, $\times 10^6$ (psi)		Sustained Modulus of Dolomite Phase E_d , $\times 10^6$ (psi) ^a	Sustained Modulus of "Matrix" Phase E_m , $\times 10^6$ (psi) ^b
1-6	2.62	2.54	2.71	2.68	2.61
1-8	1.73	1.64	1.82	2.68	1.48
12-9A	2.37	2.26	2.49	2.68	2.26
12-9B	1.91	1.80	2.02	2.68	1.56
12-U	2.26	2.18	2.34	2.68	2.14
27-4	2.17	2.11	2.23	2.68	2.07
B-A	2.68	2.51	2.86	2.68	2.68
K-B	2.15	2.07	2.24	2.68	1.86
Mo	1.13	0.94	1.32	2.68	0.62

^aThe value of sustained E_d for dolomite is assumed to be the same as the sustained E of sample B-A, which is virtually all dolomite.

^bThe values of sustained E_m were calculated by Eq. 3 using the data for the volume of dolomite, g , from Table 2.

cause it was not found to be expansive after soaking for 8 weeks in 1N NaOH. (The material did begin to expand, however, after approximately 40 weeks of continuous soaking in the NaOH solution.)

Physical and Mechanical Properties

The absorption and specific gravities of the rocks were determined by standard ASTM procedures (27) and are given in Table 2. Porosities were also determined by use of a mercury porosimeter and are given in Table 2.

The expansion of carbonate rocks against restraint involves time-dependent stresses, which in turn involve the effects of creep. Creep is normally taken into account by using a "modulus of deformation" or a "sustained modulus" value for E in calculations. In an effort to measure the materials' performance characteristics under sustained loads, stress-strain data on the rocks were obtained by applying deadweight loading to the 0.45×1 -in. (1.14×2.54 -cm) cylindrical specimens. By using a soil consolidation device, 40-kg loading increments were applied to the rock specimens. Strain readings were taken after each loading increment had been on the specimen for approximately 20 seconds. During this lapse of time some additional deformation occurs that is greater for the weaker structured rocks than for the stronger. Thus, in either case, lower than normal modulus values are obtained due to the higher strain measurements—and probably also due to the small specimen size. The order of magnitude of the values, however, is of little consequence since the study was concerned only with relative differences between the materials.

From the stress-strain data taken on 15 specimens for each rock sample, the slope of the line of best fit through the data points was determined by the method of least squares. The slopes, which are the values of the sustained modulus, are given in Table 3.

Hansen (22) has reviewed the various theoretical and practical formulas that have been proposed for correlating the modulus of elasticity of a two-phase material to the modulus of elasticity of the component materials. For a two-phase material where the modulus of the embedded particles is greater than that of the surrounding matrix, i.e., $E_g > E_a$, it was concluded that the following relationship is sufficiently accurate for most practical purposes:

$$E = \frac{1}{\frac{1-g}{E_a} + \frac{g}{E_g}} \quad (3)$$

where E = modulus of elasticity of a two-phase material and g = fractional volume of the embedded particles. For two-phase materials such as concrete, E_g is normally greater than E_a ; if the carbonate rocks included in this study are defined as two-phase materials with an E_g of the embedded dolomite rhombohedrons equal to that of the reference sample B-A, then E_a can be calculated from Eq. 3. The sustained E_a were calculated from the sustained modulus values for E and E_g and are included in Table 3.

Unrestrained Expansion Measurements

For each rock sample, three of the conically tipped specimens described earlier were tested for expansion. With the exception of the specimen size and the intervals at which length change measurements were taken, the method of test and equipment conformed to the requirements of ASTM C 586-66T. The length change measurements were taken at weekly intervals for the first 8 weeks and at 4-week intervals thereafter for a total of 20 weeks.

Restrained Expansion Measurements

Samples 1-6, 1-8, 12-9A, 12-9B, K-B, and Mo were tested for expansion in a stainless steel restraining device (Figure 5). Electrical resistance strain gauges were mounted on the top side of each device to measure the amount of strain caused by the expanding rock specimen. By use of calibration curves developed for each device, the

strain readings could be converted to the restraining forces developed by the expanding specimens or to the amount of linear expansion against restraint. Two levels of restraint were used, and they were governed by the thickness of the upper beam, which was either $\frac{1}{4}$ or $\frac{1}{8}$ in. (0.635 or 0.318 cm) thick.

For each rock sample, four specimens were mounted in individual restraining devices and placed in a polyethylene container of 1N NaOH solution (19). Sufficient solution was added to cover the rock specimens, and the strain gauge lead wires were connected to multichannel switching units. Strain readings were taken weekly for a period of 90 days.

RESULTS

The average results of the unrestrained expansion tests are shown in Figure 6. The most expansive sample, Mo, expanded an average of 7.8 percent, followed by 12-9B at 6.25 percent, 12-9A at 2.23 percent, and 1-8 at 1.52 percent, after 20 weeks in the NaOH solution. Of the expansive rock group, sample 1-6 expanded the least at 0.31 percent, with samples K-B and 27-4 expanding slightly more. The expansion of sample 12-U was insignificant. Although this rock had a relatively high dolomite content, the petrographic analysis indicated that the dolomite occurred in localized clumps—possibly interlocked with sparry calcite to form a rigid structure. It is probable that this sample would expand considerably if soaked for a longer period of time.

In general, expansion of the rock samples during the 20-week period was related to the volume of dolomite in the rock. When consideration is given, however, to the expansions of samples K-B and B-A, it is apparent that factors other than dolomite content are involved. K-B, for example, had a dolomite content virtually equal to that of sample 12-9B but expanded only a fraction as much (0.36 percent in 20 weeks). Sample B-A had a 94 percent dolomite content but did not expand during the test period. In the latter case, the massive structural fabric, with great interlocking and coarseness of the mineral constituents, probably resists alkali penetration and expansion. Of the two lithologies studied in the case of K-B, neither contained the clearly defined discrete dolomite rhombs like those shown for 12-9A. In one K-B section studied petrographically much of the dolomite was partially locked in by sparry calcite.

Somewhat similar situations existed for samples 27-4 and 12-U except that these had isolated bands or pockets of material that contained the discrete dolomite rhomb type of texture.

The expansions of alkali-reactive carbonate rocks are thus not simply related to the volume of dolomite but rather to a complex interplay of other physical variables as well. Swenson and Gillott (9), for example, have shown that rocks having small dolomite rhombs with high specific surface tend to expand the greatest over a given period of time. Porosity, as shown in Figure 7, is generally higher in the most expansive rocks, and, as can be detected from Tables 2 and 3, the sustained modulus of elasticity generally decreases with an increase in porosity. A more pronounced decline in E_n than in E occurs with an increase in porosity, indicating, as would be expected, that the pores are concentrated in the matrix phase of the rocks. One would also expect that expansion would decrease with an increase in the rigidity of the rock matrix material, and this was found to be generally the case, as shown in Figure 8. Accordingly, two carbonate rocks with equivalent chemical compositions might be expected to have vastly different expansive characteristics if their structural fabrics differ to such an extent that their elastic properties are quite different.

Given the basic constituents typical of reactive carbonate rocks, it becomes more apparent from the preceding relationships that other physical properties should also be indicative of the expansive nature of these rocks. In tests on a large number of different types of carbonate rocks, Nikolayev (28) found porosity to increase as compressive strength decreased. In still other studies on large numbers of carbonate rocks, Chel'tsov (29) found that compressive strength generally decreases with a decrease in specific gravity. Generally, expansion should increase with decreased compressive strength, and a decrease in compressive strength should be accompanied by a decrease in specific gravity. Since absorption is related to porosity, it is reasonable to expect

Figure 5. Typical device used for restraining expansive carbonate rock specimens.

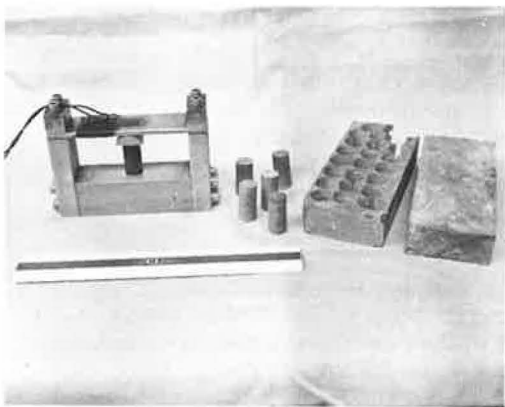


Figure 6. Average expansions of carbonate rock samples.

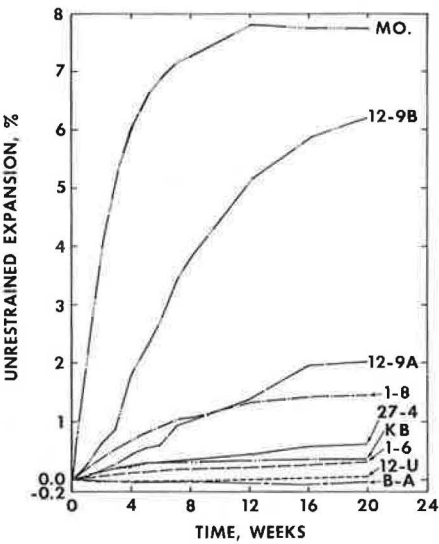


Figure 7. General increase of expansion as initial porosity increased.

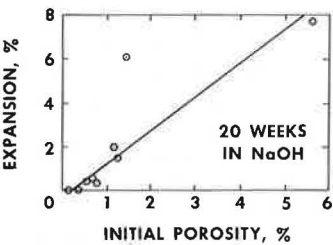


Figure 8. General decrease of expansion as sustained modulus of the rock "matrix" increased.

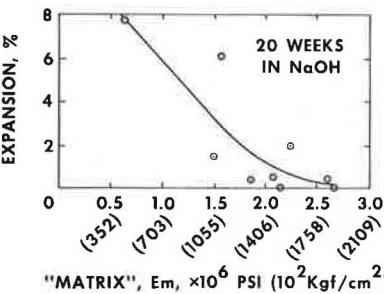


Table 4. Calculated expansions for dolomite phase of the expansive rocks.

Sample Designation	Ratio of Phase Moduli E_d/E_s	Expansion, ξ_d , Selected for Calculation Purposes (%)	Time Required to Expand 0.31% in 1N NaOH (days)	Fractional Volume of Dolomite, g	Ratio ξ_d/ξ_s	ξ_s	$\xi_s^{(b)}$, Eq. 2 (%)	Reduction in ξ_s Due to Restraint (%)
1-6	0.975	0.31	140	0.158	0.158	1.98	1.15	41.3
1-8	0.552	0.31	12	0.324	0.392	0.79	0.61	22.6
12-9A	0.843	0.31	23	0.289	0.308	1.01	0.69	31.9
12-9B	0.582	0.31	9	0.446	0.51	0.608	0.496	18.4
27-4	0.772	0.31	45	0.194	0.21	1.475	0.976	33.8
K-B	0.694	0.31	64	0.443	0.485	0.64	0.505	21.1
Mo	0.231	0.31	1.1	0.586	0.702	0.442	0.417	5.7

a highly expansive carbonate rock to generally have higher absorption, higher porosity, lower specific gravity, lower compressive strength, and a lower modulus of elasticity than less expansive rocks. A comparison of the samples based on data from Tables 2 and 3 indicates that, relative to the sample group, the most expansive rock sample, Mo, and the nonexpansive B-A both fit these criteria. The rock samples in between these two extremes generally follow suit, but uniform relationships cannot be expected since expansion depends basically on the volume and disposition of the dolomite present in the materials. In the final analysis, however, it is apparent that the sustained modulus of elasticity tends to reflect the net effect of the various physical properties of a rock.

Textural Restraint

The restraining effects of the texture of rock samples B-A and 12-U have been discussed in light of their structural properties and lack of expansion at 20 weeks in the 1N NaOH solution. If Eqs. 1 and 2 are applied to the remaining expansive samples, the effects of textural restraint on the expansion of the dolomite phase of these materials can be calculated. The use of these equations, in effect, considers textural restraint and the sustained moduli of elasticity of the materials to be synonymous. As a basis for comparison of the effects of textural restraint on the expansion of these rock samples, Table 4 has been prepared. The calculated values of ξ_s and $\xi_{s(r)}$ in the table are based on a ξ_b of 0.31 percent. This value was chosen because sample 1-6 expanded the least (0.31 percent) at the end of 20 weeks and it can be reasonably compared with identical expansion magnitudes of the other samples. The time required to attain a ξ_b of 0.31 percent ranged from 1.1 days for sample Mo to 140 days for sample 1-6. These two samples represent, respectively, the lowest and highest ratios of E_n/E_g . By comparing the samples in this manner the time factor can be eliminated. From the calculated values of ξ_s and $\xi_{s(r)}$, a percent reduction of ξ_s due to the restraint of the matrix phase of the materials was determined. If these reductions are plotted against the total sustained modulus of the rocks as in Figure 9, it can be seen that the greater the total sustained modulus of the materials is, the greater will be the reduction in the expansion of the dolomite phase. Consequently, the textural restraining effect on sample 1-6 is greater than 7 times that of sample Mo, and a corresponding effect occurs with regard to the remaining samples. Thus, a reduction in the expansion of the dolomite phase would result in a reduction in the volume change of the total material.

The results shown in Figure 9 are those calculated for the actual properties of the materials and consequently represent materials containing different volumes of expansive dolomite. For a better concept of the independent effects of textural restraint, a simulated curve is shown in Figure 10. This curve is based on the same ξ_b of 0.31 percent used previously, but it is assumed that all the rock samples contain an equal g of 0.35, while the ratios of E_n/E_g are those of the actual materials. For the particular conditions used in the calculation of this theoretical curve, it is apparent that the textural restraining effect on sample 1-6 is greater than 3 times that of sample Mo. The relationship given in Figure 10 might thus explain the relative difference in the magnitude of expansion exhibited by some carbonate rocks having nearly identical chemical compositions.

Restrained Rock Expansions

The effect of external restraint on the expansion of some selected test rocks is shown in Figure 11. The average results of three specimens subjected to the higher degree of restraint are represented by the solid curves, while one sample (1-8) subjected to the lower restraint is shown for relative comparison and is represented by the broken curve. Samples 12-9A and 12-9B expanded under restraint virtually the same as sample 1-8 and have been omitted from Figure 11 for clarity. Data for sample 27-4 were omitted due to malfunctioning of the strain gauges during testing.

If only the higher restraint specimens shown in Figure 11 are considered, it is apparent that the differences between the magnitudes of expansions of the various rock samples are not as great as those obtained for the unrestrained expansions shown earlier.

Figure 9. Effect of textural restraint on the expansion of the dolomite phase, ξ_g , of carbonate rock samples.

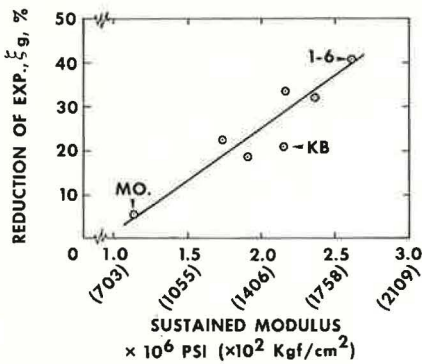


Figure 10. Theoretical curve showing the effect of textural restraint on simulated rocks having constant g and E_m/E_g equivalent to the actual rock samples.

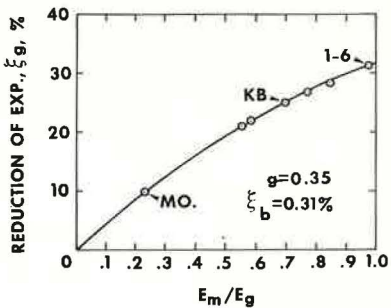


Figure 11. Restrained expansions of several carbonate rocks.

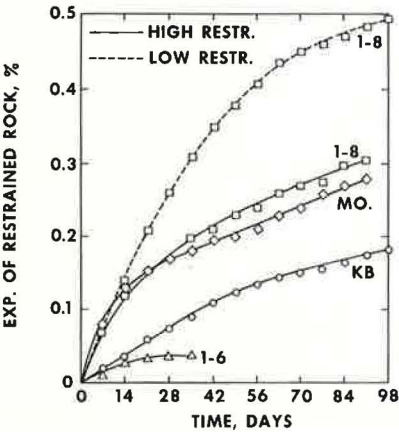
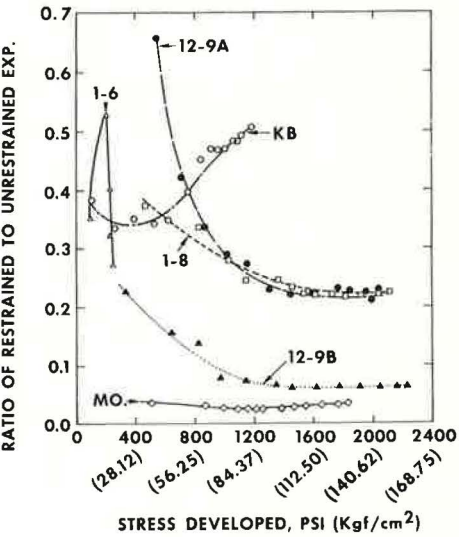


Figure 12. Stress developed by several carbonate rocks expanding against restraint.



The unrestrained expansion of sample Mo, for example, was 4 to 5 times greater than the expansions of samples 12-9A and 1-8; but when restrained, sample Mo expanded slightly less than 12-9A and 1-8. Compared with sample K-B, Mo expanded 24 times as much when unrestrained but only $1\frac{1}{2}$ times as much when restrained. Samples 12-9A, 12-9B, 1-8, and Mo had substantially different expansive characteristics when unrestrained but had relatively the same expansive characteristics when restrained. This suggests that the rigidity of a carbonate rock and the volume of dolomite contained in the materials will have considerable influence on the degree to which it will expand under restraint. Hence, a carbonate rock with low textural restraint might be highly expansive in the unrestrained NaOH test, but when restrained in a mass of concrete (or mechanically as in this experiment) it would be more compressible and therefore less expansive. Optimum combinations of textural properties and dolomite content would thus result in the most detrimentally expansive rocks in concrete. Thus, sample K-B has considerable textural restraint acting to restrain free rock expansion, but this same rigidity combined with a high dolomite content is apparently sufficient to overcome moderate external restraint. Indications from the limited data available on sample 1-6 are that the external restraint combined with a high textural rigidity and low dolomite content (15.8 percent) are sufficient to limit restrained expansion to approximately 0.037 percent. If it is assumed that the 1-6 curve would have progressed with the same trend indicated in Figure 11, a restrained expansion of 0.037 percent would be about the maximum attained. This degree of expansion would be a marginal case if the rock were used as an aggregate in concrete.

Perhaps the most conspicuous indication from Figure 11, as compared with Figure 6, is that the magnitude of restrained expansion is not proportional to the magnitude of unrestrained expansion. This is clearly shown in Figure 12, where the ratio of restrained to unrestrained expansion is plotted against the stress developed by the expanding specimens. With the exception of 1-6, which was probably restrained from further expansion, the samples with the higher ratios (considering the right end points of the curves) are in inverse order as compared with their order of unrestrained expansion. While concrete distress in the form of cracking does not always correlate with expansion, these data suggest that some rocks that expand on the order of 0.3 percent to 1 percent might be just as detrimental to concrete as the more highly expansive ones. Sample K-B, for example, was capable of developing high stresses in comparison to the relatively low magnitude of unrestrained expansion developed. In fact, the trend of the curves in Figure 12 suggests that, if a higher degree of restraint had been used in the experiment, K-B might have developed stresses of magnitudes comparable to those of 1-8, 12-9A, Mo, and 12-9B.

For the particular degree of restraint used in this experiment, stresses on the order of 2,200 psi (15 200 kPa) were developed by some samples, as is indicated by the curve for sample 1-8 in Figure 12. Since the tensile strength of concrete is normally less than 1,000 psi (6900 kPa), there is little doubt that the expansive carbonate rocks can exert pressures of sufficient magnitude to cause fracture systems in concrete. Expansive rocks like that of sample 1-6, with low dolomite content and high textural restraint, however, might be exceptional cases.

SUMMARY AND CONCLUSIONS

The equations presented in this paper can be used to estimate the magnitude of carbonate rock expansion (obtained from tests similar to ASTM C 586-66T) that would be necessary to cause certain degrees of expansion in concrete specimens containing certain volumes of the rock as an aggregate. The amount of dilution of an expansive aggregate (with an inert material) that would be necessary to keep concrete expansions within certain limits can also be estimated.

An analogy between the physical structure of portland cement concrete containing expansive aggregate particles and dolomitic carbonate rocks containing expansive dolomite rhombohedrons was drawn by considering each as a two-phase system. A select group of carbonate rocks was investigated experimentally and by combining the experimental with theoretical data. The following conclusions were drawn:

1. The expansion of reactive carbonate rocks generally increases as the sustained modulus of elasticity decreases, and increased expansion is more pronounced when the modulus of the matrix phase of the rocks decreases.

2. The calculated reduction in the expansion of the dolomite phase of the rocks investigated increased with an increase in the sustained modulus of elasticity of the materials.

3. The textural restraining effect on the least expansive of the actual rock samples investigated was greater than 7 times that of the most expansive sample. However, if all the rock samples investigated had equal dolomite contents, but the same elastic properties as the actual materials, the textural restraining effect on the least expansive rock would still have been greater than 3 times that of the most expansive sample.

4. The magnitude of restrained carbonate rock expansion is not proportional to the magnitude of unrestrained expansion, but, analogous to the effects of textural restraint, the dolomite content and the sustained modulus of the material are related to the degree of expansion under restraint.

5. The experimental data suggest that certain combinations of textural properties and dolomite content would result in the most detrimentally expansive carbonate rocks in concrete. On the other hand, rocks having low dolomite content and high textural rigidity might be relatively innocuous.

6. For the particular degree of restraint used in the experiments described in this paper, stresses on the order of 2,200 psi were developed by the expansion of some of the rock samples. It is apparent that the use of higher restraint or longer duration of test would have resulted in higher forces being exerted by some of the rock specimens.

7. Since the tensile strength of concrete is normally less than 1,000 psi, forces of the magnitude measured in these experiments show that expansive carbonate aggregates can exert pressures of sufficient magnitude to ultimately cause fracture systems in concrete made with cements having high alkali content.

ACKNOWLEDGMENTS

This research was conducted in cooperation with the FHWA and financed with HPR funds. The project was under the general supervision of the late T. E. Shelburne and J. H. Dillard, state highway research engineers.

The suggestions and guidance contributed by H. H. Newlon of the Virginia Highway Research Council and F. C. McCormick of the University of Virginia are gratefully acknowledged. Because of the nature of the subject matter, the study could not have been completed without the petrographic and chemical analysis data provided, respectively, by M. A. Ozol and W. C. Sherwood.

REFERENCES

1. Hadley, D. W. Alkali Reactivity of Carbonate Rocks: Expansion and Dedolomitization. HRB Proc., Vol. 40, 1961, pp. 462-474.
2. Sherwood, W. C., and Newlon, H. H., Jr. Studies on the Mechanisms of Alkali-Carbonate Reaction: Part I—Chemical Reactions. Progress Report 7, Potentially Reactive Rocks. Virginia Highway Research Council, Feb. 1964.
3. Lemish, John, and Moore, W. J. Carbonate Aggregate Reactions: Recent Studies and an Approach to the Problem. Highway Research Record 45, 1964, pp. 57-71.
4. Mather, Katharine, Buck, Alan D., and Luke, Wilbur I. Alkali-Silica and Alkali-Carbonate Reactivity of Some Aggregates From South Dakota, Kansas, and Missouri. Highway Research Record 45, 1964, pp. 72-109.
5. Swenson, E. G., and Gillott, J. E. Alkali-Carbonate Rock Reaction. Highway Research Record 45, 1964, pp. 21-40.
6. Hadley, David W. Alkali Reactivity of Dolomitic Carbonate Rocks. Highway Research Record 45, 1964, pp. 1-20.
7. Feldman, R. F., and Sereda, P. J. Characteristics of Sorption and Expansion Isotherms of Reactive Limestone Aggregate. Jour. ACI, Proc. Vol. 58, No. 2, Aug. 1961, pp. 203-214.

8. Gillott, J. E. Mechanism and Kinetics of Expansion in the Alkali-Carbonate Rock Reaction. Division of Building Research, National Research Council, Canada, Research Paper No. 22, 1965.
9. Swenson, E. G., and Gillott, J. E. Alkali Reactivity of Dolomitic Limestone Aggregate. Magazine of Concrete Research, Vol. 19, No. 59, June 1967.
10. Newlon, H. H., Jr., and Sherwood, W. C. An Occurrence of Alkali-Reactive Carbonate Rock in Virginia. HRB Bull. 355, 1962, pp. 27-44.
11. Swenson, E. G., and Gillott, J. E. Characteristics of Kingston Carbonate Rock Reaction. HRB Bull. 275, 1960, pp. 18-31.
12. Axon, E. O., and Lind, Junior. Alkali-Carbonate Reactivity: An Academic or a Practical Problem. Highway Research Record 45, 1964, pp. 114-125.
13. Smith, P. Learning to Live With a Reactive Carbonate Rock. Highway Research Record 45, 1964, pp. 126-133.
14. Hadley, David W. Alkali-Reactive Carbonate Rocks in Indiana: A Pilot Regional Investigation. Highway Research Record 45, 1964, pp. 196-221.
15. Dolar-Mantuani, L. Expansion of Gull River Carbonate Rocks in Sodium Hydroxide. Highway Research Record 45, 1964, pp. 178-195.
16. Hsu, Thomas T. C. Mathematical Analysis of Shrinkage Stresses in a Model of Hardened Concrete. Jour. ACI, Proc. Vol. 60, No. 3, March 1963, p. 373.
17. Timoshenko, S. P., and Goodier, J. N. Theory of Elasticity, 2nd Ed. McGraw-Hill, New York, 1951, pp. 356-359.
18. Axon, E. O. A Method of Estimating the Original Mix Composition of Hardened Concrete Using Physical Tests. Proc. ASTM, Vol. 62, 1962, pp. 1068-1080.
19. Hilton, M. H. The Effects of Textural and External Restraints on the Expansion of Reactive Carbonate Rocks. Progress Report 7B, Potentially Reactive Carbonate Rocks. Virginia Highway Research Council, 1968.
20. Newlon, H. H., and Sherwood, W. C. A Study of Remedial Methods for Reducing Alkali-Carbonate Reaction. Progress Report 4, Potentially Reactive Carbonate Rocks. Virginia Highway Research Council, May 1963.
21. Hirsh, Teddy J. Modulus of Elasticity of Concrete Affected by Elastic Moduli of Cement Paste, Matrix, and Aggregate. Jour. ACI, March 1962.
22. Hansen, Torben C. Influence of Aggregate and Voids on Modulus of Elasticity of Concrete, Cement Mortar and Cement Paste. Jour. ACI, Proc. Vol. 62, No. 2, Feb. 1965, pp. 193-216.
23. Sherwood, W. C., and Newlon, H. H. A Survey of Reactive Carbonate Aggregates in Virginia. Progress Report 2, Potentially Reactive Carbonate Rocks. Virginia Highway Research Council, Feb. 1964.
24. Bisque, R. E., and Lemish, J. Chemical Characteristics of Some Carbonate Aggregates as Related to Durability of Concrete. HRB Bull. 196, 1958, pp. 29-45.
25. Bisque, R. E. Analysis of Carbonate Rocks for Calcium, Magnesium, Iron and Aluminum With EDTA. Jour. Sedimentary Petrology, Vol. 31, 1961, pp. 113-122.
26. Handbook of Chemistry and Physics, 41st Ed. Chemical Rubber Publishing Co., Cleveland, 1959-1960.
27. ASTM Book of Standards. Part 10, Concrete and Mineral Aggregates. 1964.
28. Nikolayev, S. V. Physical and Mechanical Properties of Basic Types of Carbonate Rocks Located in Samarskaya Luka. Izdatel'stra Akad. Nauk. USSR, Moscow, 1962, pp. 210-217.
29. Chel'tsov, V. I. Firmness and Structures of Carbonate Rocks in the Melekhovo-Fedotovsk Deposit. Academy of Sciences, USSR, Papers of the Institute of Geology of Ore Deposits, Petrography, Mineralogy and Geochemistry, No. 43, 1961, pp. 25-32.
30. Gillott, J. E., and Swenson, E. G. Mechanism of Alkali-Carbonate Rock Reaction. Jour. Eng. Geol., Vol. 2, 1969.

15 YEARS OF LIVING AT KINGSTON WITH A REACTIVE CARBONATE ROCK

P. Smith, Engineering Research and Development Branch,
Ontario Ministry of Transportation and Communications, Downsview

Shortly after a number of highway bridges had been built near Kingston, Ontario, in 1956 and 1957, alkali-carbonate rock reaction was identified as being a cause of concrete deterioration observed in the area. Shortly afterward adjacent concrete structures and pavements were built of concrete aggregate from the same dolomitic limestone formations but selected as being innocuous on the basis of short-term concrete prism expansion tests (expansion using job materials not to exceed 0.02 percent after 84 days' storage at 100 percent relative humidity). Comparing the field performance of concrete in both groups of structures for up to 15 years with the results of short-term, long-term, and accelerated laboratory tests shows that selective quarrying was a successful countermeasure. Whereas the earlier structures now show severe pattern cracking in moisture-prone exposures, those built later are free of blemish.

•IN 1972, Swenson summarized studies over the last 15 or so years into concrete deterioration attributable to the reaction of certain active minerals in concrete aggregates used in Canada with alkali components in portland cement (1). The first of these studies concerned concrete deterioration at Kingston, Ontario, where a dolomitic limestone concrete aggregate was quarried from certain lithologies in the Gull River (formerly known as Black River) formation of the Middle Ordovician system. In 1957, Swenson showed that, in this case, the observed disruptive expansion of concrete in service was the result of an alkali-carbonate rock reaction (2).

Just prior to Swenson's 1957 report, the Ministry of Transportation and Communications (then Department of Highways, Ontario) had started a major highway construction program in the Kingston area. Unwittingly, therefore, potentially reactive aggregates were incorporated in the first concrete structures built on the new highway. However, as already reported (3), both short- and long-term countermeasures were rapidly developed for the bulk of the work remaining, which involved about 250,000 yd³ (191 139 m³) of concrete in pavement and bridges.

Selective quarrying was employed to obtain from the same formation concrete aggregates that did not cause excessive expansion in concrete prisms made with the job materials and stored in the laboratory at 73.4 F (23 C) and 100 percent relative humidity. Originally the limit on tolerable expansion under these conditions with the then-available cement, which had a total alkali content in the order of 1.1 percent, was set not to exceed 0.05 percent at 84 days. As soon as some experience was available from the first screening tests, the limiting expansion criterion was reduced to 0.02 percent at 84 days, and the test method was standardized. From then on, expansions were measured of 3 × 4 × 16-in. (76.2 × 101.6 × 406.4-mm) concrete prisms, made with a standard coarse aggregate grading having 3/4-in. (19.05-mm) nominal maximum size, a fixed cement factor of 525 lb/yd³ (311.43 kg/m³) using a cement of normal, for Ontario, alkali content (0.8 to 1.0 percent). The concrete was non-air-entrained, with a 3-in. (76.2-mm) slump.

Laboratory testing in this investigation was mainly confined to measuring the length change of concrete prisms, but other exploratory tests, such as the petrographic examination of rock textures in thin sections, expansion of rock prisms, and the powder cell test, were also tried. The applicability of these other test methods to the "Kingston" type of alkali-rock reaction is adequately covered by Swenson (1) and the references cited therein. Accordingly, this paper is confined to revisiting the Kingston scene 15 years later to report on the field performance of both the concrete containing the reactive rock and that containing rock from the same quarries selected on the basis of the early-expansion criteria established for concrete prisms made and tested in the laboratory, and to relate this performance to the long-term behavior of concrete prisms under standard laboratory moist-curing.

PERFORMANCE OF CONCRETE IN BRIDGES AND PAVEMENTS

Figures 1, 2, and 3 are typical of distress to be seen in concrete containing a high-alkali cement and reactive aggregate after 15 years in service.

In no case has any of the concrete had to be replaced. The importance of the nature of the exposure of the affected concrete and the availability of moisture is obvious when making field inspections of the structures. In two circumstances pattern cracking and general deterioration is always worse: where water is channeled over the face of the concrete, as at joints, due to deficiencies in design, construction, or maintenance, or where the concrete stands with a readily available source of water at its base or back that can transpire from an exposed face.

Careful examination of structures of similar type and age on the same highway, within a few miles and with similar exposure conditions, that were built with high-alkali cement from the same mill but for which the concrete aggregate was selectively quarried, shows no sign of pattern cracking or distress whatsoever. Similarly, concrete pavements constructed with selected aggregates are free of distress. It should be noted that this particular exposure condition was considered to be potentially a very severe one, since sodium chloride used for deicing in winter could regenerate alkali in the pavement concrete if alkali-carbonate reaction occurred (3).

LONG-TERM EXPANSION OF LABORATORY SPECIMENS

The data originally reported (3) carried the expansion of concrete prisms containing aggregate from the Pittsburgh quarry at Kingston (the source of aggregate in most of the structures showing distress) to one year. The investigation of possible alkali-carbonate rock reaction involving other quarries in Ontario working the Gull River formation of the Middle Ordovician system led to the resampling and testing of the Pittsburgh quarry 0 to 24-ft (0 to 7.32-m) lift (containing the most reactive rock) in order to provide a comparative expansion yardstick (4, 5).

Expansion data are now available to the age of almost 7 years for $3 \times 4 \times 16$ -in. concrete prisms made with this reference aggregate and with low-alkali (0.45 percent) cement, with normal-alkali (0.8 to 1.0 percent) cement and with normal-alkali cement fortified by the addition of sodium hydroxide to the mix water to provide the equivalent total alkali (expressed as Na_2O) of 3 percent by weight of cement. Corresponding expansion measurements on rock prisms were also made. However, the rock prisms were made with flat rather than conical ends (ASTM C 586), and therefore it will be left to the report of the companion investigation (5) to provide a comparison between rock prism and concrete prism expansions, because, for the other quarries involved, conical-ended rock prisms (which are now standard) were measured.

Figure 4 shows the expansion of the concrete prisms to 2,300 days. In addition, because of the particular interest in establishing limiting expansions at an early age so that the test method can be of practical value for selecting "non-reactive" rock for concrete aggregate, the values up to 168 days are tabulated.

It will be noted that, with the low-alkali cement, and to a lesser extent with the normal-alkali cement, expansion at early ages is a somewhat erratic indicator of what might be expected later on. However, with the fortified-alkali (3 percent) cement the expansion is immediate, dramatic, and consistent.

Figure 1. In-water pier, Cataragui Bridge, Hwy. 401, Kingston, after 15 years.



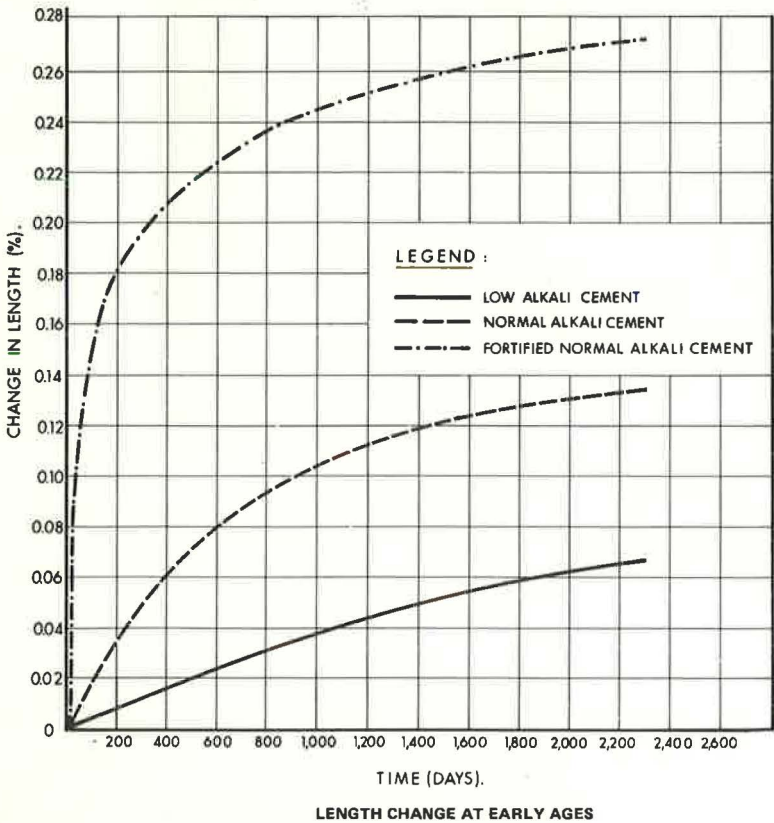
Figure 2. Abutment haunch, Sydenham Road Bridge, Hwy. 401, Kingston, after 15 years.



Figure 3. Wing wall, CPR Bridge, Hwy. 401, Kingston, after 15 years.



Figure 4. Long-term length change of concrete prisms containing reactive alkali-carbonate rock from Pittsburgh quarry, Kingston.



AGE IN DAYS *							
	14	28	56	84	112	140	168
L	+0.0025	-0.0025	-0.0025	-0.0013	-0.0013	+0.0063	+0.0064
N	+0.0038	+0.0152	+0.0165	+0.0228	+0.0266	+0.0354	+0.0392
FN	+0.0089	+0.0444	+0.0964	+0.1350	+0.1549	+0.1730	+0.1800

* Age shown is the total time period in the moist room after making & demoulding prisms. Initial zero base measurement at 7 days.

As noted in the earlier report (3), very obvious cracking is to be seen in concrete prisms when the expansion reaches between 0.10 and 0.12 percent, and minor cracking may be observed under magnification at about 0.05 percent expansion. The corresponding pattern cracking and distress that will follow within 15 years if structures are built with cement-aggregate combinations that produce expansions of this order during laboratory tests are vividly shown in Figures 1, 2, and 3.

CONCLUDING REMARKS

The investigation of alkali-carbonate reactivity of certain dolomitic limestone aggregates quarried at Kingston, Ontario, has now extended over a 15-year period.

Examination of the field performance of concrete structures incorporating reactive rock as coarse aggregate compared with that of structures incorporating rock from the same quarry that had been selectively quarried shows that rock selection has been a successful (and economical) countermeasure.

The basis established for selection of "non-reactive" rock—that is, the expansion of concrete prisms stored at 73.4 F (23 C) and 100 percent relative humidity for 84 days is not to exceed 0.02 percent—appears to be amply confirmed by the long-term performance of both concrete pavement and structures in the area. The acceleration of this test method by fortifying the normal-alkali "job" cement to 3 percent alkali is one of considerable promise. At 84 days, expansion is in the order of 6 times that occurring with the "job" materials. By no later than 21 days, a clear indication of unacceptable expansion is evident.

This investigation and its companion (5) have shown the clear benefit of involving geologists and petrographers from the beginning in the inspection and sampling of quarries and in the identification of suspect lithology by textural classification and length change of rock prisms. However, in all cases, the final engineering judgment as to the acceptability of a concrete aggregate must be based on the expansion of concrete prisms not exceeding a limit established by long-term in-service and laboratory performance.

ACKNOWLEDGMENT

This report is part of a long-term study of alkali-carbonate rock reactions in concrete in Ontario undertaken by the Engineering Research and Development Branch and the Materials and Testing Office, Construction Branch, Ministry of Transportation and Communications, Ontario.

REFERENCES

1. Swenson, E. G. Interaction of Concrete Aggregates and Portland Cement: Situation in Canada. *Engineering Journal*, Engineering Inst. of Canada, Vol. 55, No. 5, May 1972.
2. Swenson, E. G. A Canadian Reactive Aggregate Undetected by ASTM Tests. *ASTM Bull.* 226, 1957.
3. Smith, P. Learning to Live With a Reactive Carbonate Rock. *Highway Research Record* 45, 1964, pp. 126-133.
4. Ryell, J., and Smith, P. Case Histories of Poor Concrete Durability in Ontario Highway Structures. In *Performance of Concrete* (Swenson, E.G., ed.), Univ. of Toronto Press, 1968.
5. Ryell, J., Chojnacki, B., Woda, G., and Koniuszy, Z. D. The Uhthoff Quarry Alkali-Carbonate Rock Reaction: A Laboratory and Field Performance Study. Prepared for the 53rd Annual Meeting and included in this Record.

REACTION PRODUCTS IN EXPANSION TEST SPECIMENS OF CARBONATE AGGREGATE

Hollis N. Walker, Virginia Highway Research Council, Charlottesville

Detailed petrographic examinations and X-ray diffraction analyses were made of carbonate aggregates that had been tested by ASTM Methods C 227 and C 586. It was found that the most expansive mortar bars exhibited more external cracks and deposits of reaction products than did the less expansive bars. Any reaction rims were within the aggregate particles, and etching with various strengths of hydrochloric acid showed that they were negative. Rim-forming aggregates within the mortar bars appear in thin section to have undergone dedolomitization in the rim zone at the paste-aggregate interface. X-ray examination of the most expansive aggregates from these mortar bars failed to detect any brucite. However, comparison of the diffractograms generated by X-ray reflection from these reacted aggregates with diffractograms generated by the fresh, unused portions of these aggregates indicated that some type of dedolomitization reaction had occurred. The information obtained indicates that, under conditions found when testing rock for expansion by ASTM C 586, the minerals of the hydrotalcite-sjogrenite mineral groups (complex magnesium carbonate hydroxides) can be the chief products of alkali dedolomitization near the surface of the test prisms of dolomite carbonate aggregate and that brucite can be the major dedolomitization product in the interior of the aggregate.

•THE petrographic examinations described in this paper were made on samples used in a broad research program of the Virginia Highway Research Council (1). This program included studies designed to assess the ability of various test methods to detect detrimental reactions between aggregates and alkalis (2). The test methods studied included ASTM C 586 for rock prisms, ASTM C 227 for mortar bars, ASTM C 157 for concrete beams, and petrographic examinations by ASTM C 295.

Concurrently with this broad study of various test methods, the Council cooperated with the subcommittee on chemical reactions of the ASTM committee on concrete and concrete aggregates in an interlaboratory study of Method C 227 by four agencies. The purpose of this interlaboratory study was to determine the applicability of the mortar bar method to the detection of expansion caused by alkali-carbonate reactions. The results of expansion tests on specimens exposed for 6 months were reported by the four cooperating agencies in 1965 (3). A report on a portion of the study (including some petrographic examinations) conducted by the Waterways Experiment Station was submitted to the ASTM subcommittee in 1966 (4). This report included expansion results at 19 months. The major findings from the ASTM studies were summarized by Newlon (2). The petrographic examinations were made to identify products of the reactions between carbonate aggregate and alkalis in the hope that the mechanisms of alkali-carbonate reaction and expansion could be better understood.

SAMPLES

In the cooperative testing program, 5 of the 22 aggregates from the Council's broad research program were studied along with 2 aggregates submitted by the

Waterways Experiment Station, K-4 G-6 (3) and OM-16 G-1 (2). The details of the sampling for the Virginia aggregates have been given by Newlon et al. (1).

The 7 aggregates used and the percentage of expansion of the mortar bars and prisms are given in Table 1. The mortar bars had been made as part of the cooperative interlaboratory study of Method C 227 and tested in accordance with that method. The bars studied were those that had been made with high-alkali cement (Na_2O equivalent = 0.95 percent). The only aggregates that had produced significantly expanding mortar bars were 27-4, 12-9, and 1-8. The study of these 3 aggregates included petrographic examinations of matching prisms that had been tested under Method C 586.

The prisms of aggregates 27-4 and 12-9 had been shaped from pieces of the same samples of aggregates that had been used in the cooperative mortar bar testing program. In most cases the original pieces of rock from which the prisms had been cut were still available for comparison. The prisms of 1-8 had been cut from samples collected a year later than the aggregate used in the mortar bar study. The samples had been carefully chosen from the same quarry and lithologic unit, and every attempt had been made to duplicate the original 1-8 sample. The lithologic unit 1-8 is quite homogeneous; it has little variation in texture, color, or mineralogic composition.

Table 1. Samples examined for reaction products formed by the testing of aggregates for expansion due to alkali-carbonate reaction.

Aggregate ^a	Mortar Bars Examined ^b		Prisms Examined ^c			Remarks
	Percent Expansion	Months of Storage	Number of Prisms	Percent Expansion	Weeks of Storage	
27-4	0.128	25	2	1.92 0.100	200 200	Significant expansion, very faint rims Calcite dolomite Slightly dolomitic limestone
12-9	0.144	25	2	4.986 2.793 Broke after 52 weeks	200 52	Significant expansion, rims visible on polished slabs Dolomitic limestone Dolomitic limestone
29-5	0.056	25	None			Dolomitic limestone
36-X	0.064	25	None			Slightly dolomitic limestone
1-8	0.118	25	7	0.862 Too long to measure at 40 weeks 1.958 1.038 0.709 1.073 1.087 0.887	36 180 167 167 167 167	Significant expansion, no rims detectable on typical material Impure dolomitic limestone, dolomite content usually equal to calcite content
K-4 G-6 (3)	0.055	20	None			Rims very definite, micrite limestone
OM-16 G-1 (2)	0.076	20	None			Rims definite, cherty dolomitic limestone

^aThroughout the Virginia research, a 2-digit designation has been used to identify sources; the first indicates a specific quarry and the second a specific lithology. Thus "1-8" indicates a sample from quarry 1 and lithology 8.

^bHad been tested for expansion by moist storage.

^cHad been tested for expansion by storage in NaOH.

PROCEDURES

The mortar bars were examined megascopically and microscopically on exterior surfaces, fractured surfaces, cut and polished surfaces, and surfaces that had been polished and etched. Thin sections of the mortar bars and of aggregate particles from them were examined with a polarizing microscope, and particles of the aggregate were removed from the mortar bars and analyzed by X-ray diffraction procedures.

The rock prisms that had been used for expansion testing were examined in thin sections with the polarizing microscope and compared with thin sections made of the original samples from which the prisms had been cut.

One surface each of 2 tested rock prisms of each expansive aggregate was scanned with the X-ray diffractometer; an increment of thickness was ground from the surface, and the prism was scanned again. The procedure was repeated 10 times for each 27-4 prism, 7 times for each 12-9 prism, and 16 times for each 1-8 prism. In addition, one surface each of 4 additional prisms of aggregate 1-8 was scanned with the diffractometer to facilitate a detailed study of surface components. The surfaces of another prism of aggregate 1-8 were scraped in 6 very small increments until a depth of 0.001 in. (25 μ m) was removed from each side. The prisms and the scrapings were subjected to analysis by X-ray diffraction between each removal of material.

Samples of the original untreated aggregate were analyzed by X-ray diffraction.

Diffraction patterns were generated of the crystallized residue of the liquid in which the prisms had been stored.

RESULTS

Mortar Bars

Portlandite crystals and ettringite crystals were found on the surfaces and/or in the voids of all the mortar bars. Fine-grained deposits of a mixture of calcium hydroxide, sodium-calcium carbonate, and a gel-like material were found on the surfaces of all the bars. They were best developed on the bars made with aggregates 27-4 and 1-8 and least common on those made with 29-5, 36-X, and K-4 G-6 (3).

A minor amount of very fine cracks were found on the finished surfaces of the most expansive mortar bars—those containing aggregates 27-4, 12-9, and 1-8. No cracks were observed on the bars made with aggregates 29-5, 36-X, and K-4 G-6 (3).

Negative reaction rims within the aggregate were common in the mortar bars made with K-4 G-6 (3) and OM-16 G-1 (2). These rims were easily observed on etched and polished slabs of the mortar bars. Negative rims were detected only with difficulty within the aggregate particles of the mortar bar made with 12-9. A trace of rims similar to those observed on aggregate 12-9 was found on a few aggregate particles of the mortar bar made with 27-4.

Peripheral expansion cracking was easily seen on the polished surfaces of the mortar bar made with aggregate 12-9. Many of the aggregate fragments showed a narrow, dark reaction rim. Frequently a fine, white crack paralleled the reaction rim just inside it (Figure 1). Peripheral expansion cracking in carbonate aggregate is usually considered to be definite evidence of expansive alkali-carbonate reactivity. Rimmed particles of aggregate 12-9 also showed cracks that traversed the length of elongated fragments and random cracks in several directions across the more equant pieces. A few of these cracks within the aggregate were traced through the paste to the surface of the mortar bar, where they could be seen on the finished surfaces. The peripheral, random, and longitudinal expansion cracking of the aggregate is, therefore, thought to be the cause of the external cracks noted.

Aggregate 27-4 showed slight evidence of peripheral expansion cracking. No other evidence of peripheral expansion cracking was found in the aggregates of any other mortar bar.

Aggregate 1-8 exhibited a few fine, white cracks that curved across the aggregate fragment and may well have been due to expansion of the aggregate particles. Cracks of this type have been noted frequently in particles of 1-8 aggregate in concrete that has expanded in service.

Figure 1. Expansion cracking—peripheral and random—in aggregate 12-9 mortar bar.

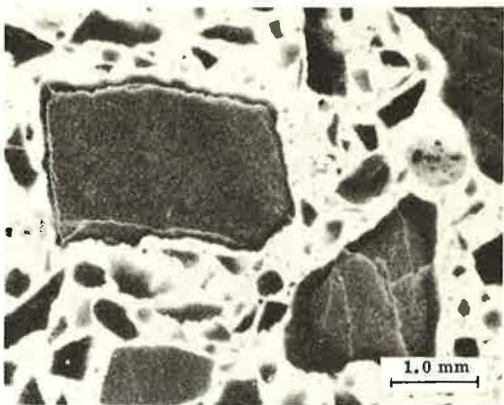


Table 2. Dolomite to calcite peaks: original samples of aggregate versus aggregate removed from mortar bars.

Aggregate	Ratio of Dolomite/ Calcite Peak Heights	
	Original Sample	After Use in Mortar Bars
27-4	1.5*	0.75
12-9	0.66	0.15
1-8	1.5	1.00

*The original aggregate 27-4 was extremely variable in composition and the ratio listed here is an average of several ratios obtained from diffractograms generated by the several types.

Figure 2. Rhombs of dolomite in thin section of 27-4 prism that show alteration of rhomb edges to fine-grained, unoriented calcite: partially crossed nicols.

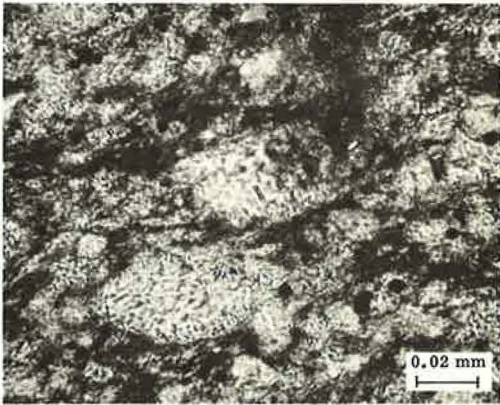
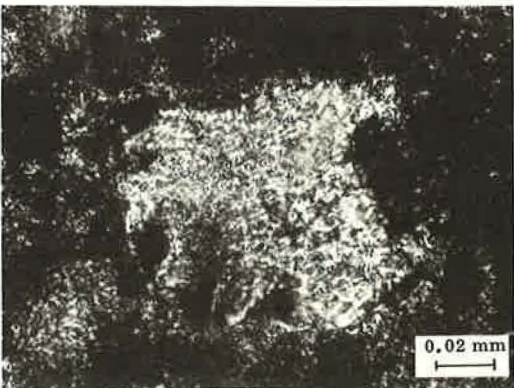
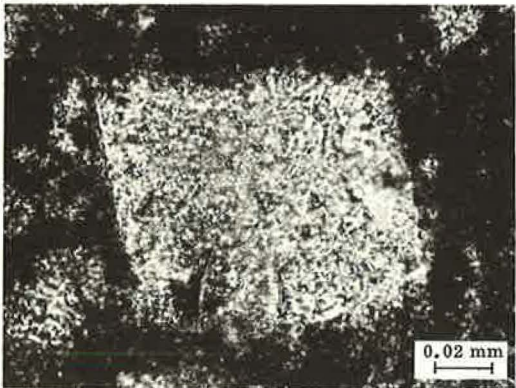


Figure 3. Large embayed rhomb of dolomite in thin section cut from prism of 1-8.



Examination of the thin sections of the mortar bars did not reveal any additional evidence that an expansive alkali-carbonate reaction had taken place. The only rims detectable in thin section were those of OM-16 G-1 (2). The innermost portion of the rim was the darkest. At the contact with the paste the aggregate particle was usually lighter in color than even the center of the particle. The rhombs of dolomite found in the outermost clarified zone of the rims exhibited an indefinite, concentric zonal structure and a wavy extinction. Wavy extinction and zoning were not present on the dolomite rhombs in the center of the aggregate particles. The aggregate apparently had undergone dedolomitization at the paste-aggregate interface.

X-ray diffraction analyses of the aggregate removed from the mortar bars were compared with diffraction analyses of the original aggregate samples, and it was seen that the dolomite-to-calcite ratio within each sample was noticeably less in the treated material than it was in the unused portion of the aggregate. The ratios are given in Table 2. No brucite was detectable in any of the samples. Despite the absence of brucite, the dolomite/calcite ratio changes indicated that a dedolomitization reaction had occurred.

Prisms

It was noted during the production of the thin sections of the prisms that had been tested by storage in NaOH that the prisms were softer and more easily ground away than were the original rock samples from which they had been cut. The surfaces of the prisms were porous, browner in color, and the details of the rock fabric were obscured.

The dolomite rhombs found in the original samples were shown in the thin sections of the expansive prisms to have been altered by the action of the NaOH. The rhomb areas often showed a wavy extinction and a zonal structure similar to that found in the reaction rims of aggregate OM-16 G-1 (2) in the mortar bars. A prism of aggregate 27-4 contained numerous rhombs of dolomite in which the outer edge of the rhomb had been altered to fine-grained, unoriented calcite (Figure 2).

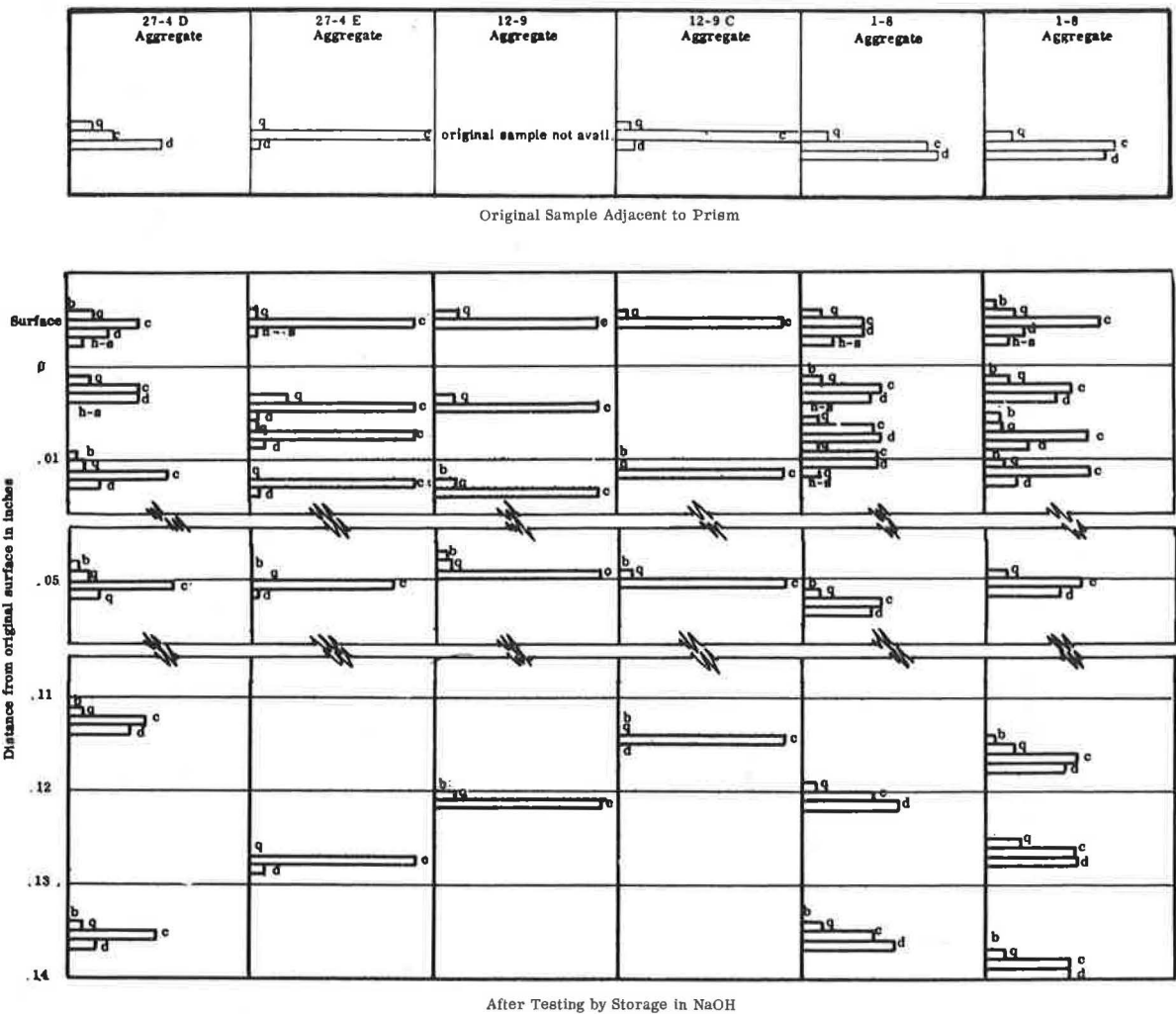
The thin section cut from the original sample of aggregate 12-9 contained a few scattered dolomite rhombs. The thin section made of the tested prism, which had been fashioned out of that sample no more than $\frac{1}{4}$ in. away, contained almost no dolomite but did exhibit rhomb-shaped holes where the dolomite had been. These rhomb-shaped voids may have contained a soft, fragile reaction product that was washed or abraded away during the production of the section. The few crystals of dolomite remaining in the prism showed a definite alteration at the edges. Prisms of aggregate 1-8 contained rhombs of dolomite with embayed edges (Figure 3).

Except for calcite, no products of the alteration of these dolomite rhombs were observed in any of the thin sections of the prisms. They may have been present interstitially in the fine-grained unoriented calcite rims or in the clay-like material occurring in the micrite matrix.

None of the thin sections made of portions of the original samples that had been immediately adjacent to the prisms showed any evidences of dedolomitization.

X-ray diffraction analyses of the surface of the tested prisms were compared with similar analyses of slabs of the original rock chunks that had been adjacent to the original prism location. Figure 4 shows the results of these analyses and the analyses of the surfaces produced by successively deeper grinding. It can be seen that the relative amounts of calcite and dolomite in the prisms were changed by the action of the NaOH. In all cases the dolomite was less abundant in the prism surface than in the original rock. Brucite was only rarely found at the surface and then in only small quantities. Four of the prisms (2 of 27-4 and 2 of 1-8) had surface reaction products that gave a broad, moderately strong reflection at 7.7 to 7.9 Å that often consisted of two recognizable peaks. No other peaks of unknown origin were clearly recognizable in scans of up to 45 deg. When the surface of the prisms was ground to a depth of 0.01 in., the 7.7 to 7.9 Å reflection was no longer present in the diffractograms. The substance causing the reflection was particularly abundant in the surface of prisms of aggregate 1-8. It was also present in small quantities in the surfaces of 12-9 prisms.

Figure 4. Change in composition with depth within the prisms.



Length of bar indicates relative height of major diffraction reflection of mineral indicated. Where there is no bar, letter indicates trace observed. Exception for surface patterns, calcite bar placed at depth at which diffractogram was generated.

b = brucite c = calcite h - s = hydrotalcite-sjogrenite mineral groups
q = quartz d = dolomite

Note: Lack of space precluded illustration of all surficial diffractograms generated of these prisms.

Five prisms of 1-8 were selected to facilitate study of this substance. X-ray diffractograms generated from the surfaces of these prisms confirmed the fact that the material with a reflection of 7.7 to 7.9 Å is frequently found in the exterior portion of prisms of 1-8 that have soaked for a long time in 1N NaOH solution. The outer 0.001 in. of one prism of aggregate 1-8 was investigated in 7 successively deeper increments. The traces of the diffractograms are shown in Figure 5. An inverse relationship may be seen between the concentration of the reaction product and the concentration of dolomite.

A composite sample of the scrapings from this prism was diluted with carbon black and burned in a visual, AC arc, grating-type spectroscope. Because of the small size of the sample the burn-off was very rapid and it was impossible to scan the entire spectrum and make a complete qualitative analysis. However, the splutter shown by the material indicated the presence of water, and it was possible to detect significant amounts of magnesium.

The information obtained from the diffraction charts was insufficient to allow a positive identification of the reaction product. When the presence of magnesium and hydroxyl groups was considered in conjunction with the original rock chemistry, the most likely cause of the diffraction peak at 7.7 to 7.9 Å was indicated to be substances of the hydrotalcite-sjogrenite mineral groups, brugnatellite, $\text{Mg}_6\text{FeCO}_3(\text{OH})_{13} \cdot 4 \text{H}_2\text{O}$, and/or pyroaurite, $\text{Mg}_6\text{Fe}_2\text{CO}_3(\text{OH})_{16} \cdot 4 \text{H}_2\text{O}$ (5, 6, 7, 8, 9, 10). If the clay minerals were broken down by the action of the hydroxide, 2 other minerals probably formed and probably contributed to this reflection. They are hydrotalcite and manasseite, both with the formula $\text{Mg}_6\text{Al}_2\text{CO}_3(\text{OH})_{16} \cdot 4 \text{H}_2\text{O}$. Because the reflection at 7.7 to 7.9 Å was broad and often consisted of two recognizable peaks, it is felt that all of these minerals may have been present.

The second strongest line expected to be produced by the minerals of the hydrotalcite-sjogrenite mineral groups is very near the calcite line at 3.86 Å. There is a definite broadening of this calcite line when the line at 7.7 to 7.9 Å is at the maximum intensity observed. This second strongest line is rather indefinite. Its presence could not be observed on diffractograms generated after the outermost surface of the prism was removed. Most of the scans did not cover the arc that includes the next strongest reflections expected. The intensities of the first 2 peaks were so low that additional peaks probably would be undetectable.

The minerals of the hydrotalcite-sjogrenite groups are closely related chemically and are in the hexagonal and rhombohedral crystal systems. They are commonly found in parallel intergrowths with each other and with brucite. They are an isostructural, polymorphous series that is probably related to the brucite group (5, 6, 7, 8).

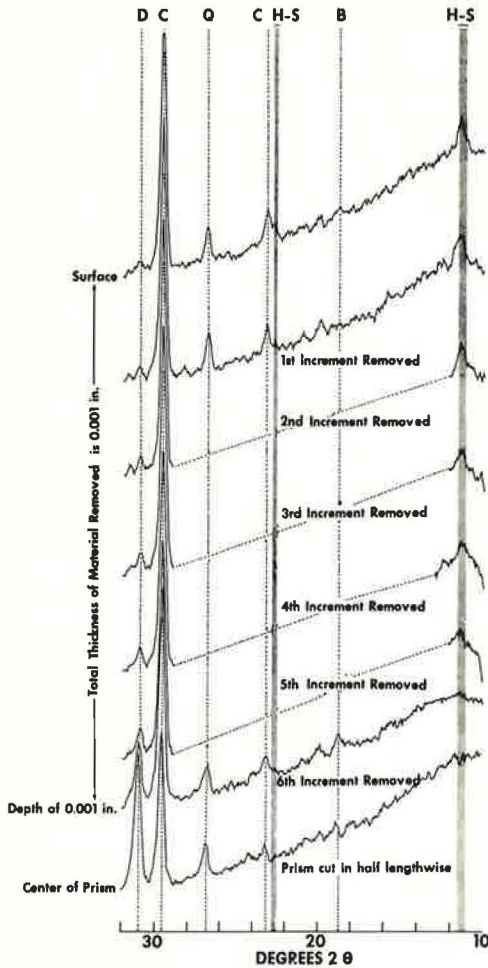
The complex carbonate hydroxide minerals have been found at the surfaces of prisms of dolomitic rocks that have been immersed in a solution of sodium hydroxide. In all cases the dolomite associated with them has been found to be less abundant than it was in the rock before it was so treated. They are apparently a reaction product of the dolomite. Brucite was present in at least trace amounts in all the dolomitic expansive prisms.

There was no apparent relationship between the amount of brucite and the depth within the prism except that the brucite was rare, especially on the surface. There was no relationship between the amount of brucite and the amount of dolomite, nor could the percent of expansion be directly related to the presence and abundance of brucite or the hydrotalcite-sjogrenite minerals. Neither the brucite nor the hydrotalcite-sjogrenite minerals occurred in the untreated samples.

Residue in Bottles Used to Store the Prisms in NaOH

The liquid in the bottles containing some of the older prisms had evaporated and carbonated, leaving a white crystalline residue. Part of this material was light and fluffy, part in needle-like crystals, and part in tabular crystals adhering to the prisms. Diffraction patterns of these substances were generated.

Figure 5. Traces of selected diffractograms produced by X-ray scan of the surfaces of prism 1-8.



- D = Expected position of dolomite reflection, 2.88 \AA "d" spacing
 C = Expected positions of calcite reflections, 3.86 \AA and 3.04 \AA "d" spacings
 Q = Expected position of quartz reflection, 3.34 \AA "d" spacing
 B = Expected position of brucite reflection, 4.77 \AA "d" spacing
 H-S = Expected positions of the reflections of the minerals of the hydrotalcite-sjogrenite groups, $7.69 - 7.93 \text{ \AA}$ and $3.88 - 3.96 \text{ \AA}$ "d" spacings

Note: Continuous scans not obtained for all the interior surfaces.

No material containing magnesium was detected, and the only definite evidence found of a reaction between the sample and the solution was a minor amount of gaylussite adhering to the prism. Minor calcite may have been present in the residue, but the sodium bicarbonate pattern obscured the calcium carbonate reflections.

FURTHER RESEARCH

More study of the reactions is needed. A method of collecting a purer and larger sample of the hydrotalcite-sjogrenite minerals should be devised so that more complete X-ray diffraction data, spectrographic analyses, and optical determinations can be obtained. Comparison with standard samples available at the Smithsonian Institution may bring a more positive identification and better understanding of the importance of these phases.

A more thorough study of the reacted surfaces and rims found on the aggregates in mortar bars and concretes should be made to ascertain whether the complex carbonate hydroxides are formed by cement alkali reaction as they are in the standard alkaline test solution.

It must not be forgotten that the rocks from which these prisms and the mortar bar aggregates were obtained are the product of natural deposition and natural organic and inorganic chemical changes. Such processes never occur uniformly throughout, but rather produce a rock that is not homogeneous but patchy in texture and composition. Therefore, it is felt that the true nature and importance of these reaction products will be ascertained only after a detailed analysis of data obtained from many more samples than were involved in this study.

ACKNOWLEDGMENTS

The generous help and encouragement provided by H. H. Newlon, Jr., are gratefully acknowledged.

This study was conducted under the supervision of former section head M. A. Ozol, begun as a part of a larger study conducted under the general direction of the late Tilton E. Shelburne, state highway research engineer, and concluded under the direction of Jack H. Dillard, present state highway research engineer. It was financed by HPR funds.

REFERENCES

1. Newlon, Howard H., Jr., Sherwood, W. Cullen, and Ozol, Michael A. A Strategy for Use and Control of Potentially Reactive Carbonate Rocks, Including an Annotated Bibliography of Virginia Research. Progress Report 8, Potentially Reactive Carbonate Rocks. Virginia Highway Research Council, June 1972.
2. Newlon, Howard H., Jr., Ozol, Michael A., and Sherwood, W. Cullen. An Evaluation of Several Methods for Detecting Alkali-Carbonate Reaction. Progress Report 5, Potentially Reactive Carbonate Rocks. Virginia Highway Research Council, May 1972.
3. Newlon, Howard H., Jr. Report of Cooperative Testing of Carbonate Aggregates for ASTM Committee C-9 Subcommittee II-b. Virginia Highway Research Council, June 1965.
4. Mather, Bryant. Information transmitted by letter to Howard H. Newlon, Jr., on portion of Cooperative Testing of Carbonate Aggregates for ASTM Committee C-9 Subcommittee II-b. Aug. 22, 1966.
5. Frondel, Clifford. Constitution and Polymorphism of the Pyroaurite and Sjogrenite Groups. *American Mineralogist*, Jour. Mineralogical Society of America, Vol. 26, May 1941, pp. 295-315.
6. Allmann, Rudolf. The Crystal Structure of Pyroaurite. Johns Hopkins Univ., July 11, 1967.
7. Brown, G., and Gastuche, M. C. Mixed Magnesium-Aluminum Hydroxides: II. Structure and Structural Chemistry of Synthetic Hydroxy Carbonates and Related Minerals and Compounds. Clay Minerals Group Meeting, Brussels, June 2, 1967.

8. Gastuche, M. C., Brown, G., and Mortland, M. M. Mixed Magnesium-Aluminum Hydroxides: I. Preparation and Characterization of Compounds Formed in Dialysed Systems. Clay Minerals Group Meeting, Brussels, June 2, 1967.
9. Ingram, L., and Taylor, H. F. W. The Crystal Structures of Sjogrenite and Pyroaurite. Mineralogical Magazine, Vol. 36, Dec. 1967.
10. Taylor, H. F. W. Segregation and Cation-Ordering in Sjogrenite and Pyroaurite. Mineralogical Magazine, Vol. 37, Sept. 1969.

DEVELOPMENTS IN SPECIFICATION AND CONTROL

Bryant Mather, Concrete Division,

U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi

Alkali-carbonate rock reactions have been regarded as capable of producing damage in concrete. Such damage may be prevented by avoiding the use of reactive rock. Rock may be classed as reactive based on its service record, its behavior in the rock cylinder test (ASTM C 586), or its behavior in concrete test specimens. Alternatively it has been suggested that damage may be prevented, as in alkali-silica reaction, by using low-alkali cement or a pozzolan or both. A supply of reactive rock may be rendered harmless by being diluted with nonreactive rock, or a source of rock may be rendered satisfactory by selective quarrying to discard reactive rock. Construction practices such as vapor barriers below pavements and more closely spaced expansion joints may assist in controlling the effects. Procedures used by the Corps of Engineers to detect reactive carbonate rock are cited and the measures to be taken are quoted.

•AT the 43rd Annual Meeting of the Highway Research Board in 1964, a symposium on alkali-carbonate rock reactions was held that was the basis of Highway Research Record 45, which contained 15 reports and a bibliography. The bibliography listed 56 items. Insofar as these dealt with specifications and control, they generally implied that aggregates of the sort that participated in the detrimental alkali-carbonate rock reaction should not be used. Presumably such exclusion would be based on service record if standard acceptance tests failed to yield a basis for rejection when results of such tests were compared with criteria such as those given in ASTM C 33. Indeed, the early work of Swenson (5) on the rock from Kingston was presented primarily in terms of the fact that the rock had, by service record, been shown to be associated with detrimental expansion of concrete but would not be detected and classed as unacceptable by then-current ASTM criteria. The work of Lemish and associates (2) in Iowa, beginning in 1957, was largely associated with explaining the unsatisfactory service of concrete made using certain sources of carbonate rock aggregates.

Swenson and Gillott (6) reported that the use of a cement of sufficiently low alkali content appeared to have provided a satisfactory solution in the Kingston area. Hadley (1) proposed the dedolomitization reaction and petrographic criteria for the recognition of types of rock that were capable of participation therein.

Newlon and Sherwood (3) reported results of a survey throughout Virginia in which rock from 7 quarries was found to expand more than 0.2 percent in the rock prism test.

This essentially was the state of the art with regard to specification and control in 1963 before the symposium.

In Highway Research Record 45, we find Hadley writing, "Simple test methods have been developed which permit rapid identification of expanding rocks and indicate their general level of reactivity." Swenson and Gillott, under the heading "Recommended Field Practice", reiterated that expansion will be reduced to "safe" values if the alkali content of the cement is held to 0.45 or less Na_2O equivalent. They also suggest dilution, attention to joints, and other details. Lemish and Moore indicated progress in detecting the features of rocks that are associated with poor service. Axon and Lind

pointed out difficulties in correlating petrographic and performance data and report that rock from one quarry was not approved because the concrete showed excessive expansion but none had yet been rejected because of expansion in the prism test. Smith (4), in presenting the results of his experience, noted that necessary precautions included (a) selective quarrying and removal of unacceptable material based on service records and tests to an age of 84 days with an expansion of 0.05 percent or more in a concrete prism made using a cement with more than 1.1 percent alkalis; (b) a vapor barrier installed below the pavement; and (c) expansion joints provided to a greater extent than normally done. Newlon and Sherwood suggested dilution and reduction of cement alkalis to 0.40 percent maximum in some cases. They proposed an expansion in concrete of 0.03 percentage points above the control as indicative of reactive aggregate.

Since 1964 there has been considerable progress in various agencies.

In ASTM Committee C-9, the rock prism test has been standardized and published as Designation C 586-66T. This says, with a variety of qualifying comments, that a rock prism that expands more than 0.10 percent is reactive rock. A letter ballot was canvassed in January 1969 on advancing this method to standard (now C 586-69, effective October 1969).

At the meeting of Subcommittee II-b of Committee C-9 in December 1968, there was discussion of publication of the results of a cooperative study of the use of the mortar-bar test (C 227) on reactive carbonate aggregates. References are now included in C 227 regarding the lower level of expansion found when this test is used on reactive carbonate rocks as contrasted with the higher levels obtained when used with an equally reactive siliceous rock.

In December 1968, Committee C-9 accepted, subject to letter ballot, a revision of the specifications for concrete aggregate (C 33) setting, for the first time, criteria on reactive carbonate rocks in the ASTM aggregate specifications. This, in effect, says a reactive carbonate rock is one that shows 0.1 percent or more expansion in 84 days or less. Since this proposal was not approved on the letter ballot, additional consideration was given to the matter. In ASTM Designation C 33-71a, effective September 1971, the topic is covered in paragraph A1.1.5, which does not include the target value previously considered, as follows:

A1.1.5 Potential Reactivity of Carbonate Aggregates—The reaction of the dolomite in certain carbonate rocks with alkalis in portland cement paste has been found to be associated with deleterious expansion of concrete containing such rocks as coarse aggregate. Carbonate rocks capable of such reaction possess a characteristic texture and composition. The characteristic texture is that in which large crystals of dolomite are scattered in a fine-grained matrix of calcite and clay. The characteristic composition is that in which the carbonate portion consists of substantial amounts of both dolomite and calcite, and the acid-insoluble residue contains a significant amount of clay. Except in certain areas, such rocks are of relatively infrequent occurrence and seldom make up a significant proportion of the material present in a deposit of rock being considered for use in making aggregate for concrete. ASTM Method C 586, Test for Potential Reactivity of Carbonate Rocks for Concrete Aggregates (Rock Cylinder Method), has been successfully used in (1) research and (2) preliminary screening of aggregate sources to indicate the presence of material with a potential for deleterious expansions when used in concrete.

Meanwhile, in the Corps of Engineers, steps were being taken to revise Appendix II, "Evaluating Reactive Aggregates", in Engineer Manual 1110-2-2000, "Standard Practice for Concrete". This appendix, as adopted in 1965, covered only materials capable of participation in the alkali-silica reaction. The revisions were begun in 1967 and modified in draft in 1968, and the manual as published in November 1971 covered these materials in its Appendix C. Because this document has not been widely circulated, Appendix C is given as an appendix to this paper.

Following the development of the criteria given in Appendix C, the selection of which was based primarily on published work done elsewhere than in the Corps of Engineers, the Waterways Experiment Station undertook an investigation having as its purpose the development of data that would verify or permit modification or refinement of the cri-

teria for permissible expansion, time, and dilution. This program is still in progress.

(Because this manuscript was prepared in 1968 for presentation at the 1969 Annual Meeting of the Highway Research Board, no attempt is made to include developments since 1968 except as they directly concern documents referenced then. Consequently, no summary or conclusion is regarded as appropriate now; none was included in the 1968 version.)

REFERENCES

1. Hadley, David W. Alkali Reactivity of Dolomitic Carbonate Rocks. Highway Research Record 45, 1964, pp. 1-20.
2. Lemish, J., Rush, F. E., and Hiltrop, C. L. Relationship of Physical Properties of Some Iowa Carbonate Aggregates to Durability of Concrete. HRB Bull. 196, 1958, pp. 1-16.
3. Newlon, H. H., Jr., and Sherwood, W. C. Initial Investigations. Progress Report 1, Potentially Reactive Carbonate Rocks. Virginia Highway Research Council, 1962.
4. Smith, P. Learning to Live With a Reactive Carbonate Rock. Highway Research Record 45, 1964, pp. 126-133.
5. Swenson, E. G. A Canadian Reactive Aggregate Undetected by ASTM Tests. ASTM Bull. 226, 1957, pp. 48-50.
6. Swenson, E. G., and Gillott, J. E. Characteristics of Kingston Carbonate Rock Reaction. HRB Bull. 275, 1960, pp. 18-31.

APPENDIX

ALKALI-CARBONATE ROCK REACTION

1. General Statement. The results of studies that have been reported indicate that four types of alkali-carbonate rock reaction may be recognized in concrete. A thorough review of research through 1964 is contained in reference 4f below. It is possible that future work will show that some of these are merely different manifestations of the same reaction, shown by different rocks under a variety of circumstances. The four types of reactions are discussed below:

a. Reactions Involving Nondolomitic Carbonate Rocks. Some rocks which contain little or no dolomite may be reactive [ref. 4a, b below]. The reaction is characterized by reaction rims which are visible along the borders of cross sections of aggregate particles; etching these cross-sectional surfaces with dilute hydrochloric acid reveals that the rims are "negative" rims; that is, the reaction rim zone dissolves more rapidly than the interior of the particle. The evidence to date indicates that the reaction is not harmful to concrete and may even be beneficial.

b. Reactions Involving Dolomite or Highly Dolomitic Carbonate Rocks. The reaction of dolomite or highly dolomitic aggregate particles in concrete has been reported [ref. 4c below]. The reaction was characterized by visible reaction rims on cross sections of the aggregate particles. When these cross-sectional areas of aggregate particles were etched with acid, the rimmed area dissolved at the same rate as the non-rimmed area. No evidence was reported that this reaction was damaging to concrete.

c. Reactions Involving Impure Dolomitic Rocks. The rocks of this group have a characteristic texture and composition. The texture is such that larger crystals of dolomite are scattered in and surrounded by a fine-grained matrix of calcite and clay. The rock consists of substantial amounts of dolomite and calcite in the carbonate portion, with significant amounts of acid-insoluble residue consisting largely of clay. Two reactions have been reported with rocks of this sort, as follows:

(1) Dedolomitization Reaction. This reaction is believed to have produced harmful expansion of concrete [ref. 4d below]. Magnesium hydroxide, brucite ($\text{Mg}(\text{OH})_2$),

is formed by this reaction; its presence in concrete which has expanded and which contains carbonate aggregate of the indicated texture and composition is strong evidence that this reaction has taken place.

(2) Rim-Silicification Reaction. This reaction is not definitely known to be damaging to concrete, although there are some data which suggest that a retardation in the rate of strength development in concrete is associated with its occurrence. The reaction is characterized by enrichment of silica in the borders of reacted particles [ref. 4e below]. This is seen as a positive or raised border at the edge of cross sections of reacted particles after they have been etched in dilute hydrochloric acid. Reaction rims may be visible before the concrete surfaces are etched. Fortunately, carbonate rocks that contain dolomite, calcite, and insoluble material in the proportions that cause either the dedolomitization or rim-silicification reactions are relatively rare in nature as major constituents of the whole product of an aggregate source.

2. Criteria for Recognition of Potentially Harmfully Reactive Carbonate Rocks. These criteria serve to indicate those dolomitic carbonate rocks capable of producing the dedolomitization or rim-silicification reaction. Since the reactions generated by some very dolomitic or by some nondolomitic carbonate rocks are not known to be harmful to concrete, no attempt is made to provide guides for recognition of these rocks at this time.

a. When petrographic examinations are made according to CRD-C 127 of quarried carbonate rock or of natural gravels containing carbonate-rock particles, adequate data concerning texture, calcite-dolomite ratio, the amount and nature of the acid-insoluble residue, or some combination of these parameters will be obtained in order to recognize potentially reactive rock. Rocks associated with observed expansive dedolomitization have been found to be characterized by fine grain size (generally 50 microns or less) with the dolomite largely present as small, nearly euhedral crystals generally scattered in a finer grained matrix in which the calcite is disseminated. The tendency to expansion, other things being equal, appears to increase with increasing clay content from about 5 to 25% by weight of the rock, and also appears to increase as the calcite-dolomite ratio of the carbonate portion approaches 1:1.

b. Samples of rock recognized as potentially reactive by petrographic examination will be tested for length change during storage in alkali solution in accordance with CRD-C 146 (ASTM Designation: C 586). Rock characterized by expansion of 0.1% or more by or during 84 days of test by CRD-C 146 shall be classified as potentially reactive.

c. If adequate reliable data are available to demonstrate that concrete structures containing the same aggregate have exhibited deleterious reactions, the aggregate shall be classified as potentially reactive on the basis of its service record.

3. Application of Criteria. The application of engineering judgment will be required in making the final decision as to which rocks are to be classified as innocuous and which are to be classified as potentially reactive. Once a rock has been classified as potentially reactive, the action to be taken should be as indicated in the following subparagraphs.

a. Avoid use as aggregate of rock classified as potentially reactive by appropriate procedures such as selective quarrying.

b. If it is not feasible to avoid the use of rock classified as potentially reactive, then specify the use of low-alkali cement, the minimum aggregate size that is economically feasible, and dilution so that the amount of potentially reactive rock does not exceed 20% of the coarse or fine aggregate or 15% of the total if reactive material is present in both.

c. If it is not practical to enforce conditions a or b, then the aggregate source which contains potentially reactive rock shall not be indicated as a source from which acceptable aggregate may be produced.

4. Literature References. The following references contain descriptions of the various types of alkali-carbonate rock reaction:

- a. U.S. Army Engineer Waterways Experiment Station, CE, Aggregate Investigations—Milford Dam, Kansas—Examination of Cores From Concrete Structures. Technical Report No. 6-629, Vicksburg, Miss., June 1963.
- b. U. S. Army Engineer Waterways Experiment Station, CE, Investigation of a Reaction Involving Nondolomitic Limestone Aggregate in Concrete, by Alan D. Buck, Miscellaneous Paper No. 6-724, Vicksburg, Miss., June 1965.
- c. U. S. Army Engineer Waterways Experiment Station, CE, Results of Laboratory Tests and Examinations of Concrete Cores, Carlyle Reservoir Spillway, Carlyle, Illinois, by W. O. Tynes, W. I. Luke, and B. J. Houston, Miscellaneous Paper No. 6-802, Vicksburg, Miss., March 1966.
- d. Hadley, D. W., "Alkali Reactivity of Carbonate Rocks—Expansion and De-dolomitization." Proceedings, Highway Research Board, vol 40 (1961), pp 462-474, 664.
- e. Bisque, R. E., and Lemish, John, "Chemical Characteristics of Some Carbonate Aggregate as Related to Durability of Concrete." Highway Research Board Bull. 196, Washington, D. C. (1958), pp 29-45.
- f. Highway Research Board, "Symposium on Alkali-Carbonate Rock Reactions." Highway Research Record No. 45, HRB Publication 1167 (1964), 244 pp.

THE UHTHOFF QUARRY ALKALI-CARBONATE ROCK REACTION: A LABORATORY AND FIELD PERFORMANCE STUDY

J. Ryell, B. Chojnacki, G. Woda, and Z. D. Koniuszy,
Ontario Ministry of Transportation and Communications, Downsview

This report, concerned with alkali-carbonate rock reaction in concrete, is a laboratory and field study of the performance of aggregate from one of the largest commercial quarries in Ontario. Laboratory studies included the fabrication of sets of concrete prisms containing a high alkali content by the addition of sodium hydroxide to the concrete mix. Field performance studies of highway structures included taking cores for petrographic examination, summarizing pattern cracking, and testing of selected structures by pulse-velocity techniques. It is concluded that the presence of approximately 20 percent reactive rock in the quarry material was responsible for widespread surface pattern cracking of the structures. Such cracking has apparently reduced the potential durability of some concrete exposed to almost continuously moist conditions but to date has not affected the serviceability of high-quality reinforced concrete. When applied, existing specifications concerned with the expansion of carbonate rocks, will in all probability produce durable concrete, although some of the symptoms of reactivity—namely, surface pattern cracking—may still occur in field concrete exposed to moist conditions.

•IN the last 15 years it has been established beyond doubt (1,2,3) that, when incorporated in concrete as the coarse aggregate, some rocks of the middle Ordovician period of the Black River (Gull) formation near Kingston, Ontario, produce excessive expansion and cracking of the concrete due to an alkali-carbonate reaction.

In the early 1960s, attention was focused on the performance of other aggregate sources in the same geological formation, in particular, the Uthoff Quarry near Orillia, Ontario. The familiar map-pattern cracking observed in the Kingston area was present in some structures containing the Uthoff aggregate, but, fortunately, this was not accompanied by the deep, wide, destructive cracks noted at Kingston. This observation of apparent deterioration in the field concrete led to a comprehensive laboratory test program started in 1963 to determine the expansion characteristics of the various layers of rock in the quarry and in two local gravel pits. The main reactive rock in the quarry was identified quickly, and a change in the method of working the quarry to exclude the reactive material permitted continued use of the Uthoff rock for concrete coarse aggregate.

Long-term data on the length change of concrete prisms made with rock from each quarry layer were compared with shorter term rock cylinder tests, and the performance of concrete in the field was observed as it related to how the quarry was worked and the inclusion, or otherwise, of reactive material.

GEOLOGY OF UTHHOFF QUARRY

The geology of the quarry was described in 1960 by Hewitt (4). The present quarry face is shown in cross section in Figure 1. The limestone of the Uthhoff Quarry, produced as coarse aggregate for concrete, from the Middle Black River (Gull) bed belongs to the same geological formation as the reactive Kingston aggregates identified in the Pittsburg and Frontenac Quarries (3). The Uthhoff stone is quite distinctive; it has a dense whitish-grey appearance and extremely fine texture and tends to have a higher proportion of flat and elongated particles than most other quarried aggregates in Ontario. Characteristics of the stone are given in Table 1.

GRAVEL SOURCES

Coarse aggregate from the two gravel pits (Ennis S&G and Uren S&G) close to the Uthhoff Quarry was included in the laboratory evaluation and the field performance surveys. These two gravel sources contained large amounts of the Uthhoff type of limestone (45 to 55 percent).

CONCRETE PRISM TESTS

Stone prepared from handpicked rock pieces from each layer within the Uthhoff Quarry was used in three sets of prisms for testing. Layer U4D consisted largely of shale and was not sampled.

Specimens with the suffix L contained a low-alkali (0.46 percent) cement (calculated as $\text{Na}_2\text{O} + 0.658 \text{ K}_2\text{O}$). Specimens N and F contained a typical Ontario normal portland cement (type 1) with an alkali content of 0.96 percent. In addition, for specimens F, sodium hydroxide was added to the concrete mix water in an amount to produce an alkali content in the mix equivalent to 3 percent by weight of the cement.

Prisms were also tested for the top lift of the Pittsburg Quarry at Kingston, the two gravel pits close to the Uthhoff Quarry, and a control nonreactive dolomite aggregate from Nelson Quarry at Burlington. In addition, prisms were tested using material from two production stockpiles of the Uthhoff Quarry rock. One sample (UAF) represented the whole 36-ft (11-m) face above the quarry floor; the other (UAP) represented material above layer U2.

The fine aggregate was an Ontario natural sand known to be nonreactive. Aggregates were oven-dried on arrival at the laboratory and saturated in water for 24 hours prior to incorporation in the concrete batch.

Concrete mixes were non-air-entrained, with a cement content of $525 \pm 5 \text{ lb/yd}^3$ ($311 \pm 3 \text{ kg/m}^3$) and slump of $3 \pm \frac{1}{2} \text{ in.}$ ($76 \pm 13 \text{ mm}$).

Each set of prisms consisted of three $3 \times 4 \times 16\text{-in.}$ ($76 \times 102 \times 406\text{-mm}$) specimens with a $\frac{1}{4}\text{-in.}$ (6-mm) diameter stainless-steel measuring pin at each end. Immediately after being cast, the concrete prisms were covered with a layer of polyethylene film and placed in the moist room (73 F, 100 percent relative humidity). The prisms were taken out of their molds the day after casting, measured for length, and returned to the moist room. Additional length measurements were made at 3, 7, and 28 days and periodically up to 7 years. The average length change, in percent based on the length at 7 days, for each set of three prisms is summarized in Figure 2.

ROCK CYLINDER TESTS

Specimens were prepared from handpicked rock samples taken at 6-in. (15-cm) intervals from layers U2 and U3, and at each 12-in. (30-cm) interval from the remainder of the Uthhoff Quarry face. Testing was carried out in accordance with ASTM C 586 except that the included angle on the conical ends was 90 deg. One test consisted of three rock cylinders. Rock cylinders were measured for length change 7 days after reaching constant length and at regular intervals for 1 year thereafter (Figure 3).

DISCUSSION OF CONCRETE PRISM TESTS

The long-term expansion criteria for nonreactive rocks in a moist-cured concrete

Figure 1. Section of Uhthoff Quarry.

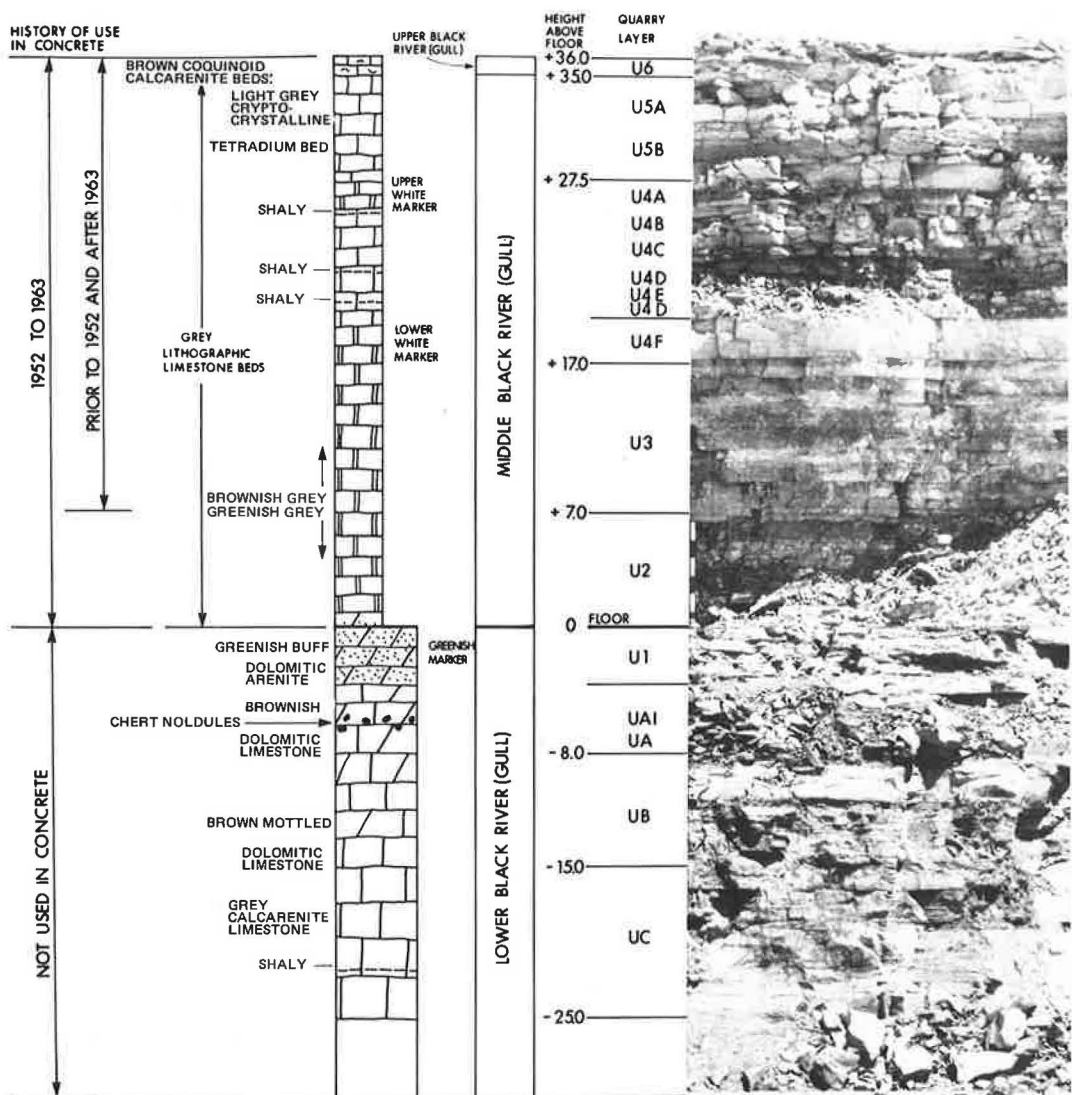


Table 1. Characteristics of rock samples from Uhthoff Quarry.

Quarry Layer	Calcite (percent)	Dolomite (percent)	Insoluble Residue (percent)	Clay Mineralogy	
				Illite	Chlorite
U6	95.6	2.8	1.6	Major	Major
U5A	90.2	4.6	5.2	Major	Minor
U5B	94.2	2.3	3.5	Major	Minor
U4A	96.4	1.8	1.8	Major	Trace
U4B	95.4	2.3	2.3	Major	Minor
U4C	94.2	2.8	3.0	Major	Trace
U4D	96.1	2.3	1.6		
U4E	93.8	3.2	3.0	Major	Minor
U4F	96.3	2.3	1.4	Major	Trace
U3	92.1	3.7	4.2	Major	Major
U2	83.3	7.4	9.3	Major	Trace
U1	10.8	44.2	45.0	Major	Trace
UA1	59.9	37.1	3.0	Major	Major
UA	64.9	31.7	3.4	Major	Minor
UB	91.4	6.0	2.6	Major	Major
UC	88.8	3.7	7.5	Major	Minor

Figure 2. Length change of concrete prisms with time.

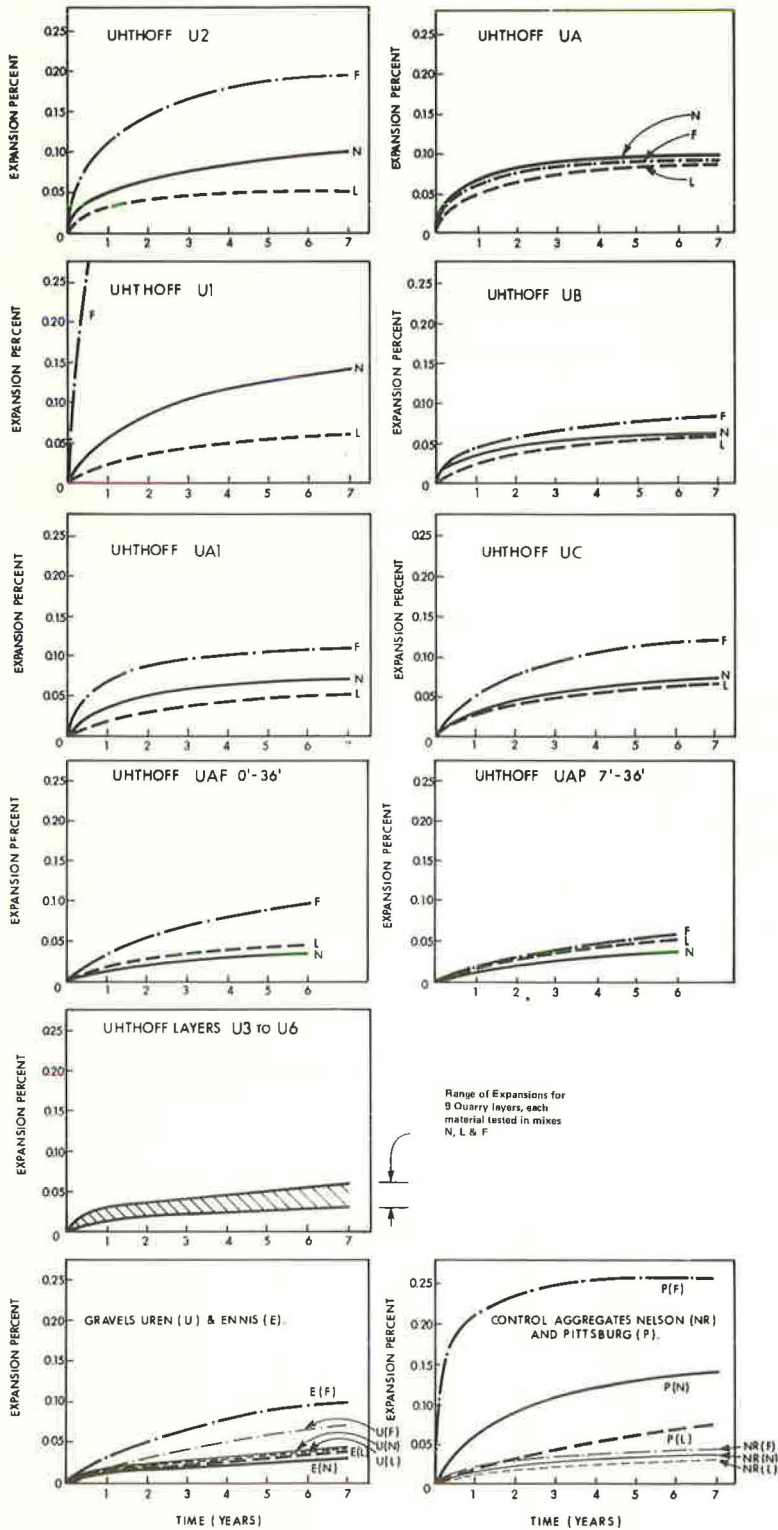


Figure 3. Length change of rock cylinders from Uththoff Quarry at 16 weeks and 1 year.

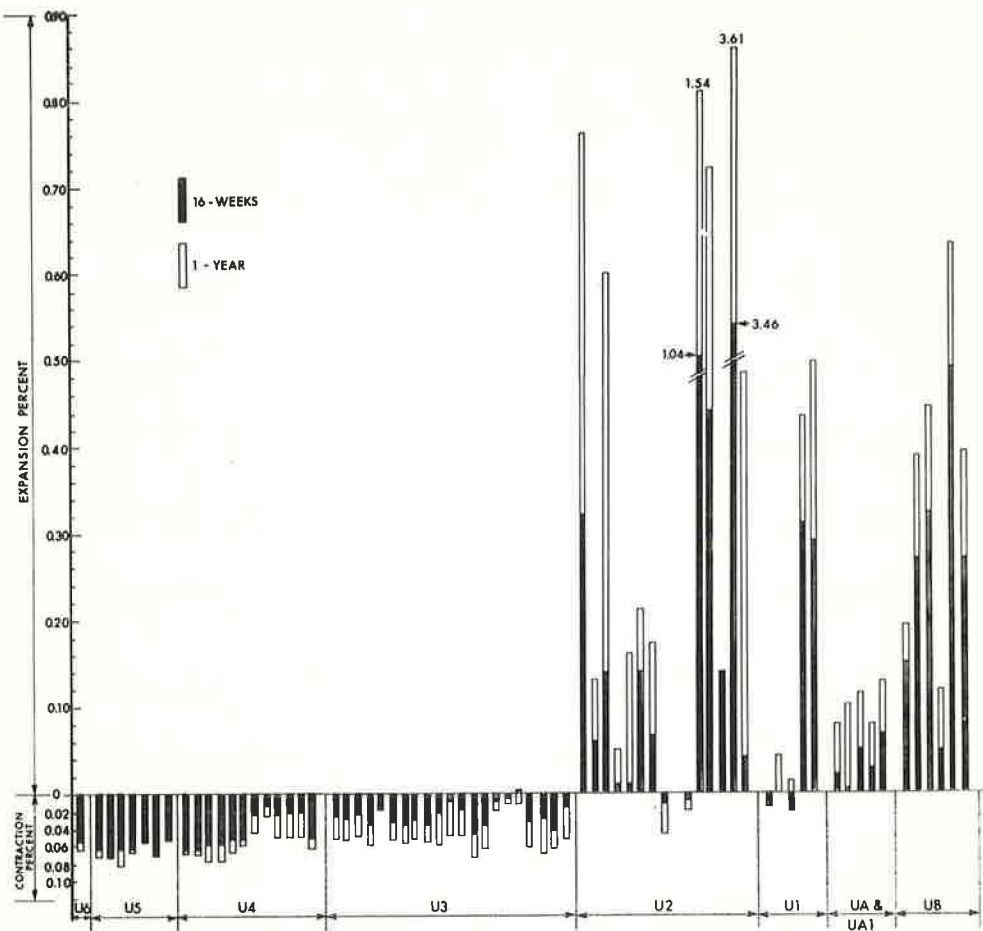


Table 2. Characteristics of rock samples from layer U2 of Uththoff Quarry.

U2 Layer	Calcite (percent)	Dolomite (percent)	Insoluble Residue (percent)	Absorption (percent)
15* (top)	83.3	6.9	9.8	0.29
14	86.1	6.9	7.0	0.15
13*	87.7	6.9	5.4	0.18
12	87.2	6.9	5.9	0.10
11	85.9	7.4	6.7	0.13
10	79.5	10.1	10.4	0.15
9	87.1	7.4	5.5	0.13
8	91.6	4.1	4.3	0.05
7	90.9	4.6	4.5	0.08
6	89.3	6.0	4.7	0.07
5*	84.9	8.7	6.4	0.20
4*	83.0	8.3	8.7	0.13
3	79.5	9.2	11.3	0.28
2*	53.7	27.1	19.2	0.52
1* (bottom)	23.2	54.3	22.5	2.00

* Indicates substantial expansion in rock cylinder test.

prism test suggested by Newlon et al. (5, 6) is 0.03 percent maximum at 1 year and 0.05 percent maximum at 5 years. This was equaled or exceeded only by each of the six Uthoff Quarry layers below U3 and material from the Pittsburg Quarry when tested with a normal-alkali cement (0.96 percent). Applying the current MTC standard of 0.02 percent maximum at 12 weeks, only three of these six Uthoff layers and the Pittsburg Quarry could be identified as reactive. The production samples UAF and UAP, the two gravel pits, and the Nelson Quarry are nonreactive by any of the above criteria.

Used with a low-alkali cement (0.46 percent), the materials in the Uthoff Quarry below U3 and the Pittsburg Quarry would generally meet the criteria for a nonreactive rock with the exception of Newlon's 5-year requirement of 0.05 percent maximum, which in most cases is slightly exceeded.

Data analysis to correlate prism expansion (N and F mixes) at various early ages, with 5-year expansions of the N mixes, did not indicate that a single early test criterion can be developed that will identify all the expansive aggregates found in this study. Linear regression analysis using 4- and 12-week data to predict 5-year expansion resulted in correlation coefficients between 0.77 and 0.86 when using data from both reactive and nonreactive materials and between 0.77 and 0.89 when using data from only expansive aggregate mixes. In general, early F beams (fortified alkali mix) were no better than early N beams (normal alkali cement) in predicting long-term expansions. No single expansion limit applied to the F and N beams at an age of 4 weeks to 1 year would reject all the reactive material while at the same time accepting all the rock identified as nonreactive.

The F prisms containing a reactive aggregate in all cases but one expanded at a much faster rate than the prisms containing the normal-alkali cement. This is particularly evident in the case of the highly expansive aggregates U2, U1, and Pittsburg. The nonreactive rock from layer U3 and above in the Uthoff Quarry and from the Nelson Quarry displayed generally little difference in prism expansions regardless of the alkali content of the concrete mix.

The performance of the F prisms containing a portion of reactive materials is interesting. The two gravel sources containing approximately 50 percent Uthoff-type limestone and the Uthoff Quarry sample UAF containing the reactive layer U2 all displayed increased expansion in the F prisms, particularly after 1 year. Thus, although a decision on the acceptance of an aggregate source at an early age cannot be based on the performance of the F prisms alone, fabrication of such specimens has a number of potential advantages. Test data allow early rejection, perhaps as soon as 4 weeks, of aggregates with a high rate of expansion in concrete. If the F prisms expand at a noticeably faster rate than the N prisms, this indicates the presence of some reactive material in the sample and provides a warning that the test period of the N prisms should be extended before making a final decision on the acceptability of the aggregate.

Most highway structures in Ontario built with concrete containing the Uthoff Quarry limestone were constructed when the working of the quarry face included approximately 20 percent of reactive material, i.e., the layer U2. Although such dilution of reactive to nonreactive aggregate has apparently produced an acceptable material, in that the expansion of specimens UAF, representing the whole 36-ft (11-m) face of the quarry, is well within the criteria for nonreactive aggregate, the field surveys revealed that pattern cracking on the concrete surface of the type often associated with alkali-concrete reactions is a common occurrence. Thus, although carbonate rocks that meet existing tentative concrete prism expansion specification requirements will in all probability produce durable concrete, some of the symptoms of alkali-carbonate reactivity may still occur in such concrete when exposed to continuously moist conditions.

DISCUSSION OF ROCK CYLINDER TESTS

Specimens of nonreactive material from layers above layer U2 underwent contraction throughout the test. Generally the amount of contraction increased with time.

Most rock cylinders from layer U2 and below had expanded substantially by 1 year. The rate and degree of expansion varied considerably. Using the criteria of 0.1 percent expansion at 12 weeks or 0.2 percent at 16 weeks, 14 out of the 31 rock samples tested

are reactive. The 0.2 percent requirement at 16 weeks appears somewhat more tolerant than the 12-week requirement. At 2 weeks significant expansion had occurred in only half the samples identified as expansive at a later age.

Most specimens with significant expansions at 1 year could be identified on the basis of a 0.1 percent expansion at 12 weeks, but there were exceptions. For example, the cylinders obtained from the bottom of layer U2 expanded 0.03 percent at 12 weeks, 0.16 percent at 6 months, and 0.49 percent at 1 year. Conversely, some layers that exhibited substantial expansion after 12 weeks displayed almost constant length for the remainder of the test.

If a criterion of 0.2 percent maximum expansion at 16 weeks is used for a nonreactive rock, the rock cylinder test, in general, predicts the behavior of the aggregate in the concrete prisms. The material identified in the concrete prism test as nonreactive is nonreactive in the rock cylinder test. More than one-third of the rock cylinders from the reactive layers U2, U1, and UB indicate the aggregate to be reactive. Rock cylinders were not tested from layer UC.

In layer UA, subdivided into UA without chert and UA1 with chert, the concrete prisms showed substantial expansion, although in the case of UA1 it is a slow mechanism. However, the five sets of rock cylinders from these layers did not produce significant expansions and normally would be labeled as nonreactive. An extension of the rock cylinder test beyond 1 year probably would have produced additional useful data on the expansion mechanism of the rocks from layers UA and UA1.

Rock cylinders were not tested from the Pittsburgh and Nelson Quarries and from the two gravel pits referred to in this study because a very large number of test specimens would be needed to properly evaluate each aggregate source.

PETROGRAPHIC STUDY

Handpicked samples of rock from the Uthoff Quarry were subjected to numerous tests, which included chemical analysis and studies of thin and polished sections.

The U2 layer is the most unusual because it contains nonreactive to extremely reactive beds. The layer has, in general, a very fine matrix of calcite intermixed with clay in which small euhedral crystals of dolomite are "afloat". The rock contains from 4.1 percent to 54.3 percent dolomite and from 4.3 percent to 22.5 percent insoluble residue. It varies from limestone to dolomitic limestone (Table 2). The reactive layers have over 5 percent dolomite and contain at least 2 percent clay, which when present as illite is sufficient to trigger the expansion mechanism.

The U2 layer is underlain by the U1 layer, which, apart from the noted expansion, is not acceptable for use as a concrete aggregate because of its poor physical characteristics. The matrix of beds UA and UA1 is very fine-grained and tends to have zonal concentrations of larger sized dolomite crystals with, apparently, sufficient amounts of clay to be reactive.

Layer UB has a very fine calcitic matrix with small crystals of dolomite and a texture similar to the other reactive rocks. It expanded to unacceptable levels but less than other reactive layers. Rock from this layer with larger dolomite crystals showed only moderate expansion in the rock cylinder test, the difference in expansion characteristics presumably being due to its heterogeneous character.

Layers U3 to U6 are characterized by dense fine-crystalline limestone containing up to 5.2 percent insoluble residue but no dolomite crystals and are considered as non-reactive.

A review of the literature on alkali-carbonate reactive rocks indicates that they are argillaceous dolomitic limestones, extremely fine-grained, particularly the calcite matrix. In general, this description also applies to the reactive rocks from the Uthoff Quarry. The dolomitic crystals, which are almost perfect rhombohedrons with a maximum size of 0.25 mm, have a tendency to be concentrated in some spots or lenses and quite often are associated with clayey or shaley seams or stylolites. This characteristic is probably responsible for the wide variation in expansion of cylinders cut from the same rock specimen that was sometimes experienced in this project.

EXAMINATION OF CONCRETE PRISMS AND FIELD CORES

Microscopic examination of laboratory prisms and field cores did not reveal a widespread network of cracks, although some of the specimens had undergone considerable expansion. The formation of new minerals and features of the alkali reaction process were in some cases difficult to determine because of the specimen preparation. Reactive features were more easily observed on fractured faces.

Prism U2-F, containing aggregate from the U2 layer, exhibited very narrow hairline surficial cracks. On a polished cross-sectional surface, about one-quarter of the aggregate particles contained fine disoriented cracks, with some peripheral cracks restricted to the particular aggregate particle. No aggregate reaction rims were noted, and the mortar was free of cracks. Most of the small voids were encrusted but not completely filled with calcite, and portlandite and ettringite crystals were observed. Prisms prepared with normal-alkali and low-alkali cement and aggregate from the U2 layer cracked and formed minerals in the voids but to a lesser degree.

Typical field cores (Vernon Narrows Structure, Huntsville) contained rock from the entire 36-ft (11-m) Uthhoff quarry face with some aggregate from the U1 layer. About half of the aggregate particles contained minor cracks, among which radial cracks predominate. Peripheral cracks are quite common but are localized around single particles. No cracks were observed in the cement paste below the immediate surface layer. A few particles have narrow discoloration rims, and many of the small voids are encrusted with calcium hydroxide and occasional formations of microscopic crystals of ettringite and portlandite. In general, field cores exhibited similar reactive features regardless of their surface condition.

FIELD SURVEYS

In 1965 and 1966, field condition surveys were carried out at 71 structures known to contain coarse aggregate from the Uthhoff Quarry and the two local gravel pits. Structures were inspected for severity of pattern cracking, scaling, surface pop-outs, etc., with more detailed testing, including pulse velocity surveys and core sampling for petrographic examination, on several typical bridges. The riding surface of all bridge decks is a bituminous layer and, in general, the top surface of the concrete deck slab was not investigated.

Three groups of structures were established: 21 completed prior to 1952, 39 built in the period from 1952 to 1959, and 11 completed in the period from 1960 to 1963. The year 1952 is significant because records indicate that only the top 30 ft (9.1 m) of the quarry (i.e., above U2) was worked prior to that time. The year 1959 was arbitrarily chosen as the date after which there is reasonable assurance that the structural concrete is properly air-entrained. Pattern cracking, which may be indicative of alkali-carbonate reactivity, on the 71 structures is summarized in the following:

Construction Period	No. of Structures Inspected	No. of Structures Affected by Pattern Cracking
1947-1951	21	9
1952-1959	39	27
1960-1963	11	0

In each of the 21 structures built prior to 1952 there is evidence of deteriorated concrete. The distinct pattern cracking observed on 9 structures most commonly affects retaining walls and concrete above the deck. Considering the age, exposure, and lack of entrained air in the concrete, the overall condition of the structures in this group is excellent.

In all, 23 structures containing Uthhoff rock and 16 structures built with local gravels represent the 1952 to 1959 construction period. Clearly defined pattern cracking was observed on 70 percent of these structures, with exposed portions—particularly where moisture is most abundant—commonly affected, with more than 50 percent of the sur-

face pattern cracked. In most cases the cracks are shallow and have not led to general disintegration of the concrete.

The concrete in the 11 structures built after 1959 is generally in excellent condition, with very fine pattern cracking affecting only isolated areas of 4 structures.

VERNON NARROWS BRIDGE

The 9-span Vernon Narrows structure, completed in 1959 and by far the largest bridge containing Uthoff stone, affords an interesting comparison between the incidence of pattern cracking in concrete that is normally in a dry condition and concrete that is wet for much of its service life. Thus, pier 6 (Figures 4 and 5), beneath an open bridge deck expansion joint, exhibits clearly defined pattern cracking that covers from 50 to 100 percent of each exposed concrete face. However, pier 7, situated beneath a continuous portion of the deck slab, is affected by barely discernable pattern cracking over approximately 5 to 10 percent of the concrete surface. Cracking also affects poorly consolidated concrete in the south abutment wall saturated by water seeping through from the backfill.

A pulse velocity survey, consisting of 600 tests on a 1-ft (0.3-m) grid, was carried out on the north half of piers 6 and 7. The data are given in Table 3.

The tests on the columns and column top beams indicate that the concrete quality in piers 6 and 7 is good and apparently unaffected by the network of shallow surface cracks. In the partly submerged column foundations, pulse velocity test values on pier 6 are somewhat lower and more scattered than on pier 7. The significance of these lower tests, particularly the 30 percent below 12,000 ft (3658 m) per second, is not clear because the core samples indicate that the concrete below the immediate surface is sound. Continued monitoring of the condition of this concrete will indicate whether the serviceability of the column foundations is impaired.

The pulse velocity survey of the top beam of pier 7 includes approximately 10 percent of test results below 12,000 ft per second. This negative skew in the test distribution curve is reflected in the high standard deviation of 1,276 ft (389 m) per second. The lower pulse velocity values on this section of the structure are in an area affected by vertical and random cracks in the concrete that appear to be associated with the freezing of water in bolt holes during construction.

FIELD SURVEY CONCLUSIONS

Some form of pattern cracking on the surface of concrete highway structures is a relatively common occurrence in Ontario. In extreme cases, particularly with a dolomitic limestone at Kingston, expansion of the concrete has resulted in extensive deterioration.

The overall effect on the durability of the structures containing reactive aggregates from the Uthoff Quarry and local gravel pits appears to be small. Cracks in the concrete that presumably result from the alkali-carbonate reaction are confined to the immediate surface of the concrete, and, in isolated cases where the cracks are accompanied by partial disintegration of the concrete, other factors appear to be involved.

Incorporating the reactive Uthoff Quarry layer U2 in concrete aggregate from 1952 to 1963 has almost doubled the number of structures on which distinct pattern cracking is present in the concrete. Considering that many of the structures containing the U2 layer were built some time after 1952, the incorporation of the reactive layer in the concrete has increased significantly the incidence of pattern cracking. The lower pulse velocity characteristics of some concrete in pier 6 of the Vernon Narrows Bridge indicate that under continuously moist conditions some reduction in durability may occur without apparently affecting the serviceability of high-quality reinforced concrete.

CONCLUSIONS

Laboratory tests have clearly demonstrated that some lower lifts of the Uthoff Quarry contain expansive carbonate material that will ultimately produce cracks in concrete. The expansion, after 6 years of continuous moist storage, of such material,

Figure 4. Vernon Narrows Bridge, columns of pier 6 (left) and pier 7 (right). Pier 6 is situated beneath an open bridge deck expansion joint.



Figure 5. Vernon Narrows Bridge, pier 6, pattern cracking of column foundation. Black and white strip is in 1-in. increments.



Table 3. Pulse velocity tests (in feet per second).

Location	Pier 6 (wet)	Pier 7 (dry)
Column foundation		
Average	12,330	14,160
Standard dev.	589	328
Column		
Average	14,530	14,840
Standard dev.	322	266
Column top beam		
Average	14,260	13,820
Standard dev.	423	1,276

Note: 1 ft = 0.3048 m.

measured in concrete prisms made with cement containing an alkali content of 0.96 percent, varied from 0.07 percent to 0.13 percent. In contrast, a dolomite material used as the control aggregate, with a long history of excellent performance in concrete highway structures, had an expansion value of 0.04 percent.

The top 36-ft (11-m) lift of the quarry, worked for many years to produce coarse aggregate for concrete, had a 6-year expansion of 0.04 percent. Within this lift, layer U2, which constitutes approximately 20 percent of the face, exhibited an expansion of 0.11 percent.

Newlon's suggested criterion for concrete expansion of not greater than 0.03 percent at 1 year is more effective in identifying expansive rocks than the current MTC requirement of 0.02 percent at 84 days.

Expansive rocks tested in concrete prisms made with a low-alkali cement (alkali content 0.46 percent) exhibited expansions generally less than the limits noted above.

Concrete prisms containing sodium hydroxide, to produce a high alkali content (3 percent) in the mix, have a number of advantages in the evaluation of aggregate sources. The extra cost of fabricating such test specimens appears justified by results.

Rock cylinders representing expansive carbonate material in the beds of the quarry below layer U3 showed wide variations in expansion within each layer. Comparison between such specimens and the expansion of concrete prisms containing aggregate representing the whole layer is therefore difficult. Rock cylinder tests of the nonexpansive material above layer U2 exhibited contractions after 1 year, which indicates an apparent zero risk of rejecting acceptable material with this test.

A rock cylinder expansion criterion of 0.2 percent at 16 weeks suggested by Newlon identified most of the expansive layers as containing some reactive material, but some specimens with an expansion below 0.2 percent showed significant expansion after 1 year of testing.

The rock cylinder test cannot be viewed as the sole basis for rejection of material for use in concrete. Aggregate sources shown to contain potentially reactive material in this test should be evaluated in concrete prisms, with portland cements of varying alkali content where appropriate, before a decision is made on their acceptability or otherwise.

Thin-section petrographic studies of rock specimens from the quarry indicated that the expansive carbonate material—i.e., layer U2 and below—possesses the textural characteristics for reactive rocks as identified by other investigators. In general, the texture is a very fine calcitic matrix containing some clay and scattered crystals of dolomite. The layers in the Uhthoff Quarry identified in this investigation as reactive usually contain more than 5 percent dolomite and an insoluble residue content that results in at least 2 percent clay in the aggregate.

Concrete structures built with coarse aggregate that included the reactive layer U2 generally exhibited more pattern cracking than structures constructed when the reactive layers of the quarry were not worked.

The pattern cracking on exposed surfaces in most cases appears unlikely to affect the expected service life of the structures. The cracks are confined to the surface and have not produced general disintegration of the concrete matrix. Cracking is most pronounced on those portions of the structure exposed to periodic wetting, such as sidewalks, handrail posts, retaining walls, and vertical columns beneath open deck expansion joints.

The 4:1 dilution of good to expansive material that formed the normal working face of the quarry prior to 1964 produced negligible expansion of moist-cured concrete prisms in the laboratory. Nevertheless, the occurrence of pattern cracking in structures containing this material indicates a potential durability level of the concrete below what might be considered normal. Of the potential solutions to the problem, working the upper part of the quarry so as to exclude all material below layer U3 for use as concrete aggregate was deemed more satisfactory than using a low-alkali cement with rock containing some reactive material.

ACKNOWLEDGMENTS

The authors wish to thank S. Farkas, of Limestone Quarries Ltd., for providing invaluable help and understanding in this project. Special thanks go to D. W. Hadley of the Portland Cement Association, Skokie, Illinois, who carried out a program of laboratory tests that helped in the initial identification of the main reactive rock in the Unthoff Quarry, and to G. Sloan of MTC Ontario, who provided much help in aggregate testing and petrographic studies.

REFERENCES

1. Swenson, E. G. A Canadian Reactive Aggregate Undetected by ASTM Tests. ASTM Bull. 226, 1957.
2. Swenson, E. G., and Gillott, J. E. Characteristics of Kingston Carbonate Rock Reaction. HRB Bull. 275, 1961, pp. 18-31.
3. Smith, P. Learning to Live With a Reactive Carbonate Rock. Highway Research Record 45, 1964, pp. 126-133.
4. Hewitt, D. F. The Limestone Industries of Ontario. Industrial Mineral Circular 5, Ministry of Natural Resources, Division of Mines, Queen's Park, Toronto, Ontario, 1960.
5. Newlon, H. H., Ozol, M. A., and Sherwood, W. C. An Evaluation of Several Methods for Detecting Alkali-Carbonate Reaction. Progress Report 5, Potentially Reactive Carbonate Rocks. Virginia Highway Research Council, May 1972.
6. Newlon, H. H., Sherwood, W. C., and Ozol, M. A. A Strategy for Use and Control of Potentially Reactive Carbonate Rocks, Including an Annotated Bibliography of Virginia Research. Progress Report 8, Potentially Reactive Carbonate Rocks. Virginia Highway Research Council, June 1972.

BRIDGE DECK DETERIORATION PROMOTED BY ALKALI-CARBONATE REACTION: A DOCUMENTED EXAMPLE

M. A. Ozol, Martin Marietta Laboratories*, and
H. H. Newlon, Jr., Virginia Highway and Transportation Research Council

Fifteen Interstate bridges in Virginia were constructed within the same time period in the same general locality and with the same materials and engineering design, except for differences in the lithologies of the coarse aggregate. In 9 of the structures an argillaceous dolomitic limestone was used, in 6 a crystalline dolomite. Both aggregates meet abrasion and soundness specifications. The limestone has a rock prism expansion in alkaline solution of 1 percent at 6 months; the dolomite under the same conditions shows no expansion. Recent surveys of the 15 bridges, opened to traffic in July 1960, show 90 percent of the slabs of the 6 containing dolomite to be free of pattern cracking and scaling, but only 15 percent of the slabs of the 9 bridges containing limestone to be free of these defects. The 15 structures have the same exposure and maintenance history, and there are no differences in the air void system characteristics of the hardened concrete that could explain the differences in pattern cracking and scaling frequency. Petrographic examination reveals that the argillaceous dolomitic limestone has the fabric of a prototypical alkali-reactive carbonate rock, and the concrete containing this stone has abundant hairline cracks propagating through both the paste and the aggregate. The other stone is a mosaic of equigranular, interlocked dolomite subhedra, and the concrete containing it is virtually free of distress. Tests using a nonreactive aggregate as a diluent and cement with varying percentages of alkalis show increasing freeze-thaw distress with increasing alkalis or with increasing reactive aggregate at constant cement alkali content.

*ALTHOUGH the reaction between certain carbonate aggregates and the alkalis in cement has been extensively studied (1), documented reports of detrimental field performance are comparatively few. Such performance information is available from 15 bridges of the Interstate system in Virginia that were opened to traffic on July 1, 1960; all were constructed during the same 9-month period on two separate but connecting projects in the same geographic and climatic zone. The 15 were built with the same engineering design and construction techniques. All have 8-in.-thick decks on either 3 or 4 simple spans, and all were placed with ready-mixed concrete screeded and finished by hand. Type II cement from the same commercial source and natural siliceous fine aggregate from the same geologic source were used in all of the concrete. The structures are in the same highway residency and are maintained and deiced by the same personnel and equipment.

In short, everything about the concrete in the 15 bridges is the same, within the limits of construction practice, except for differences in the lithologies of the coarse

Publication of this paper sponsored by Committee on Performance of Concrete—Physical Aspects.

*This paper is based on work performed while Mr. Ozol was at the Virginia Highway Research Council.

aggregate used. The authors consider that the circumstances of the construction and location of the bridges were such that a situation akin to a controlled experiment (in which the only variable was coarse aggregate) was available by chance in the field. Nine of the bridge decks, which will be called the northern project, contain a dark, argillaceous, dolomitic limestone. Six, designated the southern project, contain a pure crystalline dolomite.

Both coarse aggregates met conventional soundness requirements and Virginia Department of Highways specifications for abrasion resistance (AASHTO T 62) of maximum loss at 400 revolutions of 35 percent and MgSO_4 soundness (AASHTO T 104) of maximum loss at 5 cycles of 8 percent. The important characteristics of these aggregates and the main requirements for the concrete are given in Tables 1 and 2 respectively.

The soundness loss of the aggregate for the northern project was higher than that for the southern (Table 1). Although both aggregates passed the tests required by the specifications, it might be argued that the sensitivity of the northern project aggregate to freezing and thawing was greater than that of the aggregate in the southern project and that the distress (described later) was due to that fact. The effects of freezing and thawing and alkali reactivity on the durability of the aggregate are discussed at the conclusion of the paper.

Rock prisms of the northern aggregate have an average expansion in alkaline solution of about 1 percent at 6 months, whereas the southern aggregate has no expansion for the same length of time. The northern aggregate was from the source that contained the stone designated "1-8" in previous Virginia Highway Research Council reports of studies of alkali-carbonate reactivity (2, 3, 4). Pertinent to the subject of expansion of the rock in alkaline solution is the fact that plant analyses supplied by the cement producer for the period of construction indicate a total alkali content between 0.45 and 0.55 percent expressed as Na_2O equivalent. In addition, data from random samplings from various construction projects during this period (Figure 1) show that 80 percent of the samples from the cement source were between 0.40 and 0.60 percent total alkalis. The remaining 20 percent were greater than 0.60 percent alkalis, although in general the cement would conventionally be referred to as "low alkali".

CONDITION OF BRIDGE DECKS

Severe scaling on some of the bridges, which subsequently necessitated periodic repairs, was noted after the second winter; however, a detailed slab-by-slab survey of the decks was made in the summer of 1966 when the decks were about 8 years old. The survey revealed two principal types of distress: scaling (Figure 2) and map or pattern cracking (Figure 3), which in most cases was coincident with the scaling. The frequency of occurrence and the amount of area affected by this distress were vastly different for the northern and southern projects. On the southern project, 90 percent of the decks were free of pattern cracking, and of the 10 percent affected all had less than 25 percent of the area affected. Some 65 percent of the decks in the southern project were free of scaling. Of those not free, 85 percent had less than 25 percent of the area affected. Figure 4 is a typical view of the pavement condition in the southern area.

In contrast, in the northern area only 15 percent of the decks were free of pattern cracking, and of the 85 percent affected more than two-thirds had over 25 percent of the area affected. Only 12 percent of the area in the northern project was free of scaling, and of the scaled fraction a little more than half had more than 25 percent of the area affected. Table 3 gives a more complete representation of the percentage of affected decks and the area of those decks affected.

EXAMINATION OF HARDENED CONCRETE

Of the total number of cores taken from bridges on both projects, 17 were designated for petrographic examination; 10 were taken specifically to represent a particular condition, either poor or good, and 7 were taken in an unbiased manner by selecting the coring locations with a table of random numbers. The cores were subjected to various examinations. Pertinent information about the cores is summarized in Table 4.

Table 1. Aggregate characteristics.

Property	Northern Project	Southern Project
Bulk specific gravity (ASTM C 127)	2.74	2.82
Absorption, percent at 24 hours (ASTM C 127)	0.32	0.50
Dolomite/total carbonate, percent	63	100
Insoluble residue, percent	28	6
Prism expansion, percent at 6 months (ASTM C 586)	+0.95	-0.02
Magnesium sulfate soundness loss, percent at 5 cycles (ASTM C 88)	5.59	2.60
L.A. abrasion loss, percent (ASTM C 131)	15.8	28.6

Figure 1. Alkali contents of cements from the same source used during the period of construction of bridges in both the northern and southern projects.

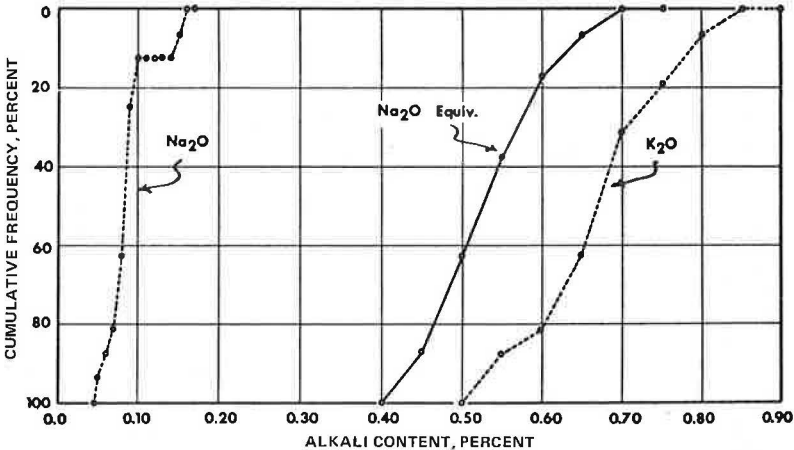


Figure 2. Typical view of severely scaled area in northern project.



Figure 3. Typical view of pattern cracking on northern project.



Figure 4. Typical view of pavement condition in southern project.



Table 2. Concrete specifications.

Property	Requirement
Cement content, minimum lb/yd ³	588
Water/cement, maximum	0.49
Slump, range, in.	0-5
Air content, range, percent	3-6
Maximum aggregate size, in.	1

Table 3. Proportion of decks affected by pattern cracking of northern and southern projects.

Description	Northern Project	Southern Project
Pattern cracking		
Percentage of decks not affected	15	90
Percentage of decks affected	85	10
Percentage of affected decks with		
>25 percent of area affected	56	0
>50 percent of area affected	27	0
>75 percent of area affected	15	0
Scaling		
Percentage of decks not affected	12	65
Percentage of decks affected	88	35
Percentage of affected decks with		
>25 percent of area affected	47	15
>50 percent of area affected	27	10
>75 percent of area affected	9	5

Air Void System

The average total void content of the hardened concrete, as determined in accordance with ASTM C 457, for random samples taken on the northern project was 3.9 percent, and for similar samples from the southern project it was 3.8 percent (Table 4). There were no significant differences in the amounts of air in the hardened concrete, in the properties of the void systems, or in the characteristics of the mortar fractions of the northern and southern concretes that would account for the difference in the performance of the two groups of bridges. Examination of the pattern cracking and scaling in a dry condition, which tends to minimize its severity, would in all likelihood lead an observer to the conclusion that it was an example of common scaling caused by freezing and thawing.

Petrographic Examination

Twelve cores from the northern and 5 from the southern project were examined petrographically using conventional techniques. Relevant features of the examinations are summarized in the following paragraphs.

Southern Project—Core samples from undistressed pavement surfaces on the southern project had finishing marks still well preserved, with strong lineation and good relief (Figure 5). The fine aggregate was firmly held and the paste was somewhat carbonated and hard and dense. There were no hairline cracks or similar features of distress. Observation of at least 4 horizontal interior polished surfaces for each core from the southern project revealed only that the paste had a few isolated, fine, discontinuous cracks that did not transect either coarse or fine aggregate particles.

The coarse aggregate used in the southern project was a typical example of a rock of its particular kind; a nearly pure "crystalline" dolomite, which is a coarse-grained mosaic of generally equigranular interlocked dolomite subhedra, with a small amount of quartz, feldspar, and opaque minerals (Figure 6). The total acid-insoluble residue comprised about 6 percent of the rock.

Almost all of the dolomite coarse aggregate particles had internal fractures and veins that were intrinsic to the stone but did not extend into the paste. An example is shown in Figure 7.

In summary, the concrete from the southern project was of good quality and exhibited no features of distress. The minor, isolated, discontinuous cracking in the paste did not connect with cracks and fractures that were intrinsically present in the coarse aggregate.

Northern Project—The concrete specimens from the northern project were taken to sample two conditions: (a) presumably "good" areas that exhibited no scaling or pattern cracking and (b) scaled and pattern-cracked areas. Pavement surfaces on core samples taken to represent the latter condition typically showed no visible lineations or finishing marks but instead displayed deep scale with a maximum relief of about 15 mm and well-exposed coarse aggregate—some particles of which were disrupted and broken apart as shown in Figure 8. There was a network of vertical-trending hairline cracks transecting, parallel to, and radial to coarse aggregate particles.

Interior polished surfaces of the core showed essentially the same crack pattern as did the pavement surface (Figure 9). If some lateral shifting with the depth of the cracks is allowed for, the surface pattern could be superimposed on the trace of the pattern on the interior surface with good agreement, and in many cases cracks could be traced down the sides of the cores to the interior surfaces. These observations established that the cracks were not isolated and discontinuous but were a continuous 3-dimensional network through the mortar and coarse aggregate particles and that the pavement surface cracks were extensions of cracks from below.

Interestingly, the interior polished surfaces of concrete cores from the so-called good areas of the northern project (Figure 10) showed exactly the same features as those from the scaled areas, the main difference between the concretes being the presence of surface scaling. These features are shown in Figures 11 and 12.

Thus for both surface conditions, i.e., scaled and unscaled, typically there were internal cracks or fractures that presumably originated from the coarse aggregate

Table 4. Air void system and petrographic information.

Core No.	Designation and Estimated Condition	Total Void Content (percent)	Entrained Air (percent <1 mm)	Specific Surface (in. ⁻¹)	Other Features
Northern Project					
0150	Specific poor, rotten	0.6	0.2	324	Hairline cracks parallel to pavement surface through paste and coarse and fine aggregate; internal cracks in coarse aggregate; incipient boundary separations; paste dull, chalky, opaque, absorptive; fracture through very few of coarse aggregate particles; water voids and entrapped air voids extremely sparse.
0151	Specific good	5.6	3.7	532	Hairline cracks in pavement surface continue below to display crack pattern in paste along which leaching has taken place; few cracks in paste in lower part of core; internal cracks in aggregate and boundary separations around coarse aggregate particles.
0152	Specific good	4.6	2.6	515	Interior aggregate cracks; some extend into paste, not well developed; aggregate-paste interface tight with few separations.
0153	Specific light scale	1.0	0.3	274	System of cracks parallel to pavement surface, noticeable boundary openings; internal cracks in aggregate, especially in upper portion of core.
0154	Specific good (best)	4.1	3.0	713	Fine cracks in surface and upper core, not abundant; minor amount of internal aggregate cracks; no well-developed paste-aggregate separations.
0171	Specific scaled	4.7	2.9	528	Few cracks in paste, especially near steel; boundary separations more abundant at bottom of core.
0172	Random good	4.5	2.5	509	Fine cracks in paste, a few emanating from reactive aggregate particles.
0173	Random good	3.6	2.1	633	Fine paste cracks and boundary separations.
0174	Specific scaled area	6.2	3.0	473	Many fine cracks in paste; many water voids.
0175	Random good	5.1	2.7	497	Poor cement-to-aggregate bond.
0176	Specific random cracking	1.5	0.3	189	
0177	Random good	5.9	3.3	616	Fine cracks in surface and upper core, not abundant; numerous internal aggregate cracks; no well-developed aggregate-paste separations.
Southern Project					
0163	Random good	3.8	2.5	699	Minor amount of very fine boundary separations at bottom.
0164	Random good	3.8	1.9	702	Fine boundary separations at bottom of core.
0165	Random good	3.5	1.6	485	Discontinuous cracks in paste at bottom of core.
0166	Specific scaled area	3.3	1.7	621	Many fine cracks in top of core; open fractures in large aggregate at bottom of core.
0170	Random slight scale	4.5	2.3	556	Some boundary separations and weak bond.

Notes: 1. The fine aggregate in all the cores is a subangular to subrounded natural sand, mostly quartz but with some feldspar and some crushed quartz particles. 2. For the northern project, the coarse aggregate is black, light and dark gray, angular to subangular, crushed dolomitic limestone, occasionally visibly argillaceous, pyritiferous, and fossiliferous. 3. For the southern project, the coarse aggregate is "crystalline" dolomite, slightly sandy, partly brecciated and recemented.

Figure 5. Typical view of pavement surface of core from southern project; note finishing marks still visible.



Figure 6. Typical view of crystalline dolomite coarse aggregate from southern project; field is 1.3 mm across, plane polarized light.

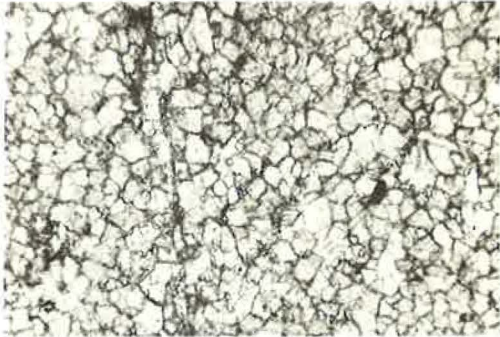


Figure 7. Detail view of coarse aggregate particle from southern project in cut and polished surface; note internal fractures and veins within the particle.



Figure 8. Top slice of core (pavement surface) from scaled and pattern-cracked area in northern project; note exposed coarse aggregate (dark gray to black) and crack across core and alongside coarse aggregate particles.



Figure 9. Interior polished surface of core with same crack pattern as the pavement surface (underside of slice in Figure 8).

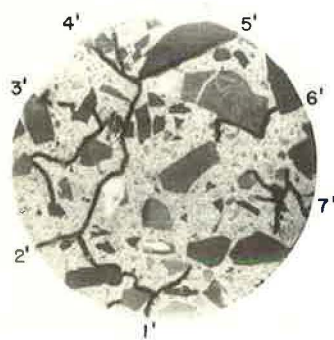


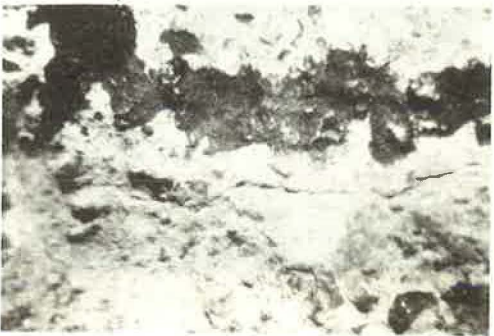
Figure 10. Core location on northern project showing healthy-looking pavement, apparently free of scaling and cracks.



Figure 11. Core from location in Figure 10 showing system of hairline cracks outlined in black.



Figure 12. Detail view of crack in Figure 11.



particles, extended into the paste (sometimes only a short distance), and dwindled to fine cracks or ended as radial cracks against other coarse aggregate particles. More often, the cracks extended through the paste a greater distance, transected coarse and fine aggregate particles, and ultimately connected with other cracks of similar origin (Figure 13).

At least some of the internal coarse aggregate cracks were intrinsic to the material, since a few (about one-tenth as many as in the aggregate in the concrete) could be found in sectioned and polished particles from stockpiles of current production. This property may be viewed as another factor of similarity between the concretes from the northern and southern projects. That is, both of the concretes had coarse aggregates that had internal fractures intrinsic to the material. But in the case of the northern stone, the expansion, presumably due to alkali reactivity, had exploited the intrinsic weakness, which was subsequently further exploited by freezing and thawing.

In any case, for the concrete of the northern project it was difficult, at best, to separate the distress due solely to reactivity from that due to freezing and thawing, but the general mechanism was almost certainly that internal fractures were produced within the aggregate (Figure 14), later extended to its edges (and eventually into the paste), propagated through the coarse and fine aggregate particles, and finally connected with cracks of a similar history.

Petrographically, the coarse aggregate used in the northern project is very like the prototypical alkali-reactive carbonate rock (ASTM C 294). As seen in Figure 15, it is composed of isolated or "floating" dolomite euhedra and subhedra in a matrix of turbid, fine-grained, partially sparry but mainly micritic calcite and indistinct material that, on the basis of acid-insoluble residue determinations, is presumably clay.

EXPERIMENTAL EVIDENCE

At the time the distress was observed it was not possible to obtain materials that would precisely represent the two aggregate sources involved. However, as part of a research program on potentially reactive carbonate aggregates, material from the quarry that supplied the northern project was extensively studied during the period that the decks were being constructed. No similar studies were made of the aggregate from the quarry that supplied the southern project because preliminary samplings of that quarry had indicated no potential reactivity. Thus direct experimental comparisons could not be made. Materials from the same vicinity and containing the same type of nearly pure "crystalline" dolomite were included in the testing program to determine resistance to freezing and thawing using ASTM procedure C 291 in concretes made with cements of varying alkali contents. The resistance to freezing and thawing of concrete with aggregate like that used in the southern project was good and unrelated to alkali content. The resistance to freezing and thawing of aggregate "1-8" was poorer and furthermore was shown to be a function of the alkali content of the cement. In that experiment the northern stone was diluted with a granite control aggregate and mixed with cements of varying alkali contents. The results of that experiment are shown in Figure 16.

Note that, after 300 cycles, for concrete with 100 percent reference aggregate, the relative dynamic modulus, P_r , is 100; but that, for instance, at 50 percent northern aggregate and high-alkali cement, P_r is about half of what it was before; and that, the lower the alkali content, the better was the performance in the freezing and thawing tests.

As noted previously, the rock prism expansion of the northern stone in an alkaline solution indicated that it was of the alkali-carbonate reactive type.

Apparently the resistance to freezing and thawing of the aggregate in concrete is related to the alkali content of the cement; the relation is that increased alkalies increase expansion from alkali-carbonate reaction and cause cracking, which in turn lowers the relative modulus.

CONCLUSIONS

The differences in the appearances and performances of the concretes in the northern and southern projects were due to the difference in the coarse aggregates used, since all other factors were the same. While the aggregate used in the northern project had

Figure 13. Detail view of coarse aggregate particle from northern project showing internal crack extending into paste and through fine aggregate particle.

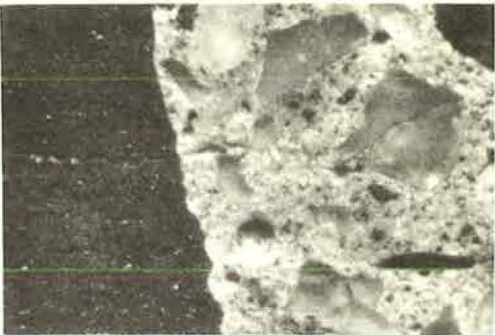


Figure 14. Family of internal cracks in coarse aggregate particle in concrete from northern project; scale divisions are 0.1 in.

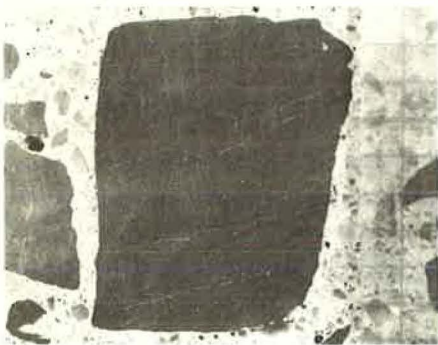


Figure 15. Detail view of argillaceous dolomitic limestone from northern project showing interstitial clay and dolomite rhombs; field is 0.3 mm across, plane polarized light.

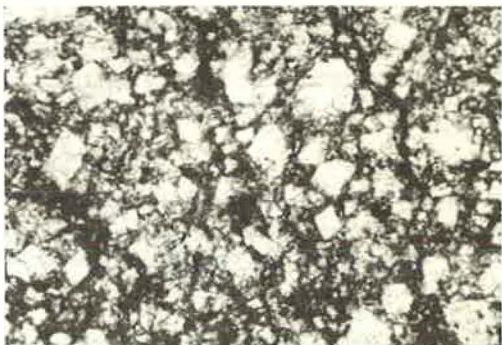
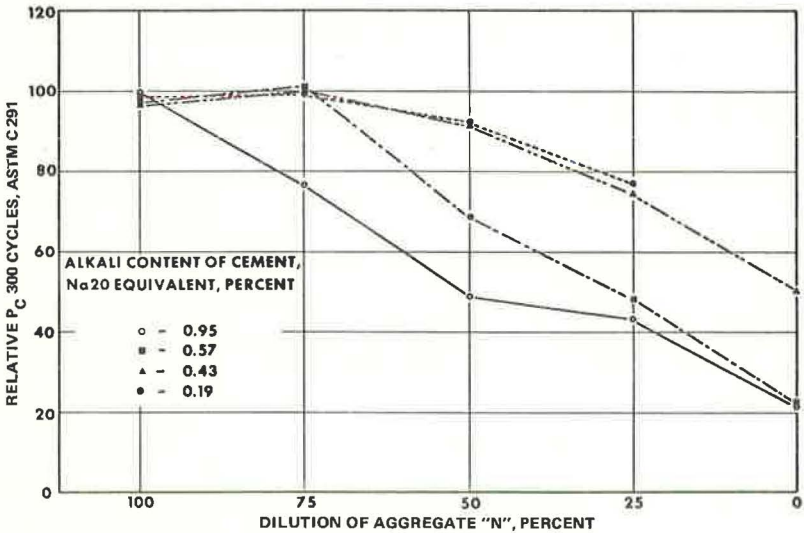


Figure 16. Freeze-thaw durability of dolomitic limestone (aggregate N) with varying concentrations of granite reference aggregate and varying percentages of alkalis.



a lower resistance to freezing and thawing, results from freezing and thawing tests (with varying dilution of reactive stone versus alkali content) suggest that alkali-carbonate reactivity, due to increasing alkali content, was a strong determinant of performance, whatever freeze-thaw sensitivity the stone may have had.

REFERENCES

1. Symposium on Alkali-Carbonate Rock Reactions. Highway Research Record 45, 1964, 244 pp.
2. Newlon, Howard H., Jr., and Sherwood, W. Cullen. An Occurrence of Alkali-Reactive Carbonate Rock in Virginia. HRB Bull. 355, 1962, pp. 27-44.
3. Newlon, Howard H., Jr., Ozol, Michael A., and Sherwood, W. Cullen. An Evaluation of Several Methods for Detecting Alkali-Carbonate Reaction. Progress Report 5, Potentially Reactive Carbonate Rocks. Virginia Highway Research Council, May 1972.
4. Newlon, Howard H., Jr., Sherwood, W. Cullen, and Ozol, Michael A. A Strategy for Use and Control of Potentially Reactive Carbonate Rocks, Including an Annotated Bibliography of Virginia Research. Progress Report 8, Potentially Reactive Carbonate Rocks. Virginia Highway Research Council, June 1972.

SPONSORSHIP OF THIS RECORD

GROUP 2—DESIGN AND CONSTRUCTION OF TRANSPORTATION FACILITIES

W. B. Drake, Kentucky Department of Transportation, chairman

Committee on Performance of Concrete—Physical Aspects

William P. Chamberlin, New York State Department of Transportation, chairman
Philip D. Cady, Kenneth C. Clear, Herbert K. Cook, Clarence E. DeYoung, Wade L. Gramling, Paul Klieger, Thomas D. Larson, Kenneth R. Lauer, William B. Ledbetter, Bryant Mather, Frederick A. Renninger, Joseph E. Ross, W. M. Stingley, V. R. Sturrupe, Harold Edward Vivian, Jukka E. Vuorinen, Richard D. Walker, E. A. Whitehurst

Committee on Performance of Concrete—Chemical Aspects

Carl F. Crumpton, State Highway Commission of Kansas, chairman
James R. Clifton, Ludmila Dolar-Mantuani, W. L. Dolch, Bernard Erlin, J. E. Gillott, David R. Lankard, Katharine Mather, Richard C. Mielenz, Maurice G. Pattengill, Robert G. Pike, Peter Smith, Neils Thaulow, George J. Verbeck, Harold Edward Vivian, Hollis N. Walker

W. G. Gunderman, Transportation Research Board staff

Sponsorship is indicated by a footnote on the first page of each report. The organizational units and the chairmen and members are as of December 31, 1973.