NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM REPORT 65

ONE-CYCLE SLOW-FREEZE TEST FOR EVALUATING AGGREGATE PERFORMANCE IN FROZEN CONCRETE

HIGHWAY RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING

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RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION
OF STATE HIGHWAY OFFICIALS IN COOPERATION
WITH THE BUREAU OF PUBLIC ROADS

SUBJECT CLASSIFICATION:
CEMENT AND CONCRETE
MINERAL AGGREGATES

HIGHWAY RESEARCH BOARD
DIVISION OF ENGINEERING NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING 1969

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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

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

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

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

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

This report is one of a series of reports issued from a continuing research program conducted under a three-way agreement entered into in June 1962 by and among the National Academy of Sciences-National Research Council, the American Association of State Highway Officials, and the U. S. Bureau of Public Roads. Individual fiscal agreements are executed annually by the Academy-Research Council, the Bureau of Public Roads, and participating state highway departments, members of the American Association of State Highway Officials.

NCHRP Project 4-3(1) FY '66
NAS-NRC Publication 1719
Library of Congress Catalog Card Number: 75-602412

FOREWORD

By Staff

Highway Research Board

The problem of determining the acceptability of aggregates for use in portland cement concrete that is to be subjected to freezing and thawing, plus other forms of exposure in cold climates, continues to plague highway agencies. Although the tests described in this report and the companion *NCHRP Report 66* are not a panacea for all of the problems associated with concrete where freezing conditions prevail, they do provide a basis for an initial screening to determine the acceptability of new aggregate sources and for identifying the particular undesirable aggregate fractions that might be removed to provide improved concrete performance. It is hoped that the suggested use of a battery of tests will permit a more discriminate use of aggregate with resulting improved utilization of current aggregate supplies and improved performance of concrete pavements and structures. Both reports will be of primary interest and value to highway materials and testing engineers. Concrete researchers and bridge engineers concerned with the problem of bridge deck deterioration should also find the report of special interest.

The durability of exposed concrete is influenced by several factors but performance records indicate that, as the predominant constituent of the mix, the qualities of the coarse aggregate significantly affect the ability of concrete to successfully resist damage caused by alternate freezing and thawing. Excessive volume change of the coarse aggregate particles is considered the primary physical factor contributing to early concrete deterioration. For this reason, the objective of NCHRP Projects 4–3(1) and 4–3(2), begun in 1963 at Virginia Polytechnic Institute and The Pennsylvania State University, respectively, was to develop a quick test or tests for identifying aggregates which undergo destructive volume change when frozen in concrete.

Currently used aggregate tests can be grouped in two general categories. Some tests are based on a correlation between some identifiable and measurable aggregate property characteristic and known field performance of the aggregate. Others depend on simulating the service environment to which the concrete will be exposed as a method of predicting field performance. Such tests can be performed either on individual aggregate particles or on the sample representative of an aggregate source.

From the practicing highway engineer's point of view, the ideal solution to the problem would involve a "black box" with dials that could be set for the environment and structure form under consideration. Aggregates would be funneled in, some sort of separation or modification would occur and out would come streams of acceptable and rejected aggregates. However, a single basic aggregate property that could be used to trigger a black box mechanism has not been found.

In view of the importance of tests both on individual aggregate particles or fractions and on the entire sample from a source, two separate studies were undertaken. The work at Virginia Polytechnic Institute under Project 4–3(1), reported herein, concentrated on devolopment of a one-cycle freeze test for predicting the durability or performance of concrete using coarse aggregates as produced from a particular source. The interim report was published as *NCHRP Report 12,* "Identification of Aggregates Causing Poor Concrete Performance When Frozen." The companion study at The Pennsylvania State University under Project 4–3(2), dealt with individual aggregate fractions, both from the standpoint of correlation with known field performance and the simulating of field environmental conditions. The Pennsylvania State University interim report was published as *NCHRP Report 15,* "Identification of Concrete Aggregates Exhibiting Frost Susceptibility."

The V.P.I. One-Cycle Slow-Freeze Test

During the V.P.I. program, a one-cycle slow-freeze test was developed and evaluated by correlating results with the durability factor obtained from ASTM Method C290 (the standard freezing and thawing test in water). The one-cycle test involves fabrication of 3 x 3 x 16-inch concrete specimens using aggregates representative of the aggregate source and, after 7-day curing, placing the specimens in a conventional household deep freeze chamber. Strain measurements are made with a Whittemore strain gauge at 5- to 15-minute intervals over a 4-hour cooling period. From the data collected, the cumulative length change is plotted vs time and the time slope, b_t, determined as the minimum slope that can be found within a ⅓-hour or greater time range.

Nineteen coarse aggregates from different parts of the country and representing varying performance histories were used to fabricate 336 specimens during the evaluation program. A sufficiently strong relationship was found between the time slope and the durability factor at 100 freeze-thaw cycles to indicate that the one-cycle test could be used as a screening test where durability factor can be considered as a measure of potential field performance. The method has the advantages of simplicity, minimal equipment requirements, and relative rapidity. It is described in detail in the report.

The Pennsylvania State University Program

Several methods of identifying frost-susceptible aggregate particles were evaluated using two approaches. First, a test method simulating field conditions was investigated with the objective of providing a test that would directly indicate probable field performance. This method was developed from the T. C. Powers' concept of a field exposure test first published in 1955. It involves the casting of specimens using the aggregate under study and, after curing, cooling at 5 F per hour while automatically recording temperatures and dilation (strain). Test cycles are run every two weeks until expansion beyond the elastic limit occurs. A means was developed to determine the period of frost immunity that could be expected for the aggregate study. The work on development and evaluation of the slow cooling method was performed with carefully selected homogeneous aggregate fractions because it was felt that beneficiation of an aggregate would be by removal of deleterious fractions. Also, only by evaluating relatively homogeneous fractions can reproducible results amenable to statistical interpretation be obtained. A complete description of the slow cooling method is contained in *NCHRP Report 66* in a generalized form so that it may be used to identify particles or fractions to be removed through beneficiation or for aggregate source acceptance testing. It requires expensive, sophisticated equipment and takes a relatively long time to perform; however, it appears to have the ability to distinguish between "marginal" aggregates—a very significant asset.

The second approach by Pennsylvania State consisted of an investigation of several rapid techniques for obtaining relative frost-susceptibility ratings of individual particles or fractions. It was found that a skilled petrographer, although untrained in concrete technology, could rate the relative frost susceptibility of homogeneous aggregate fractions by megascopic examination and could be of invaluable service in preparing aggregates for testing and in evaluating the test results. A combination of permeability coefficient and volumetric particle expansion provided good relative rating of the aggregate fractions, and vacuum saturated absorption resulted in only a slightly inferior rating.

The use of the petrographer or a single particle test to provide a "good" or "poor" rating for an aggregate fraction requires minimal equipment and time, but this approach lacks the ability to discriminate in the "intermediate" performance range. It will be useful in identifying deleterious particles that should be removed in beneficiation. For the acceptance of an aggregate source, field correlation of performance with the amount of a particular aggregate fraction under consideration must be obtained.

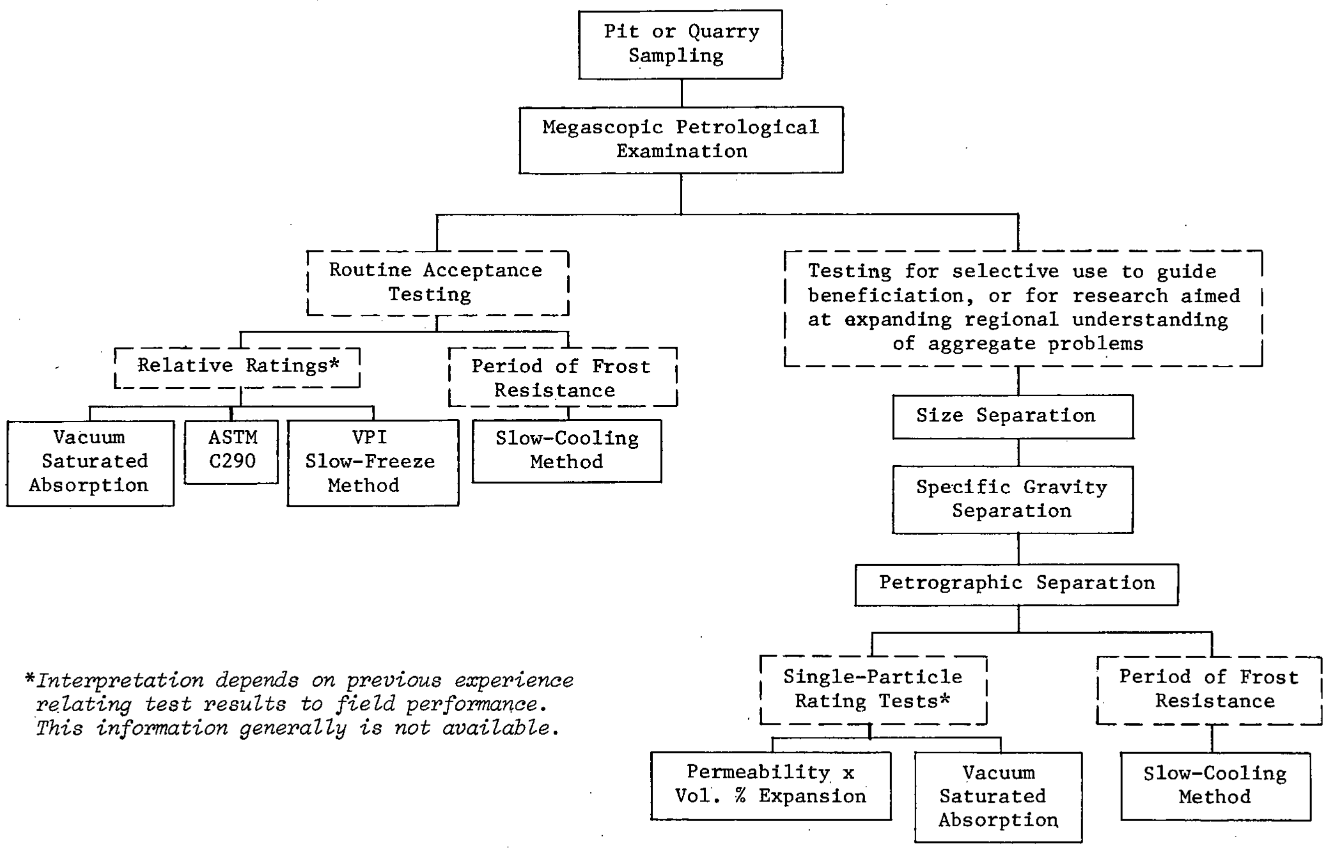
Application of Findings

On the basis of research conducted under these two projects and supported by field experience, it is apparent that no single test having the attributes of speed and simplicity is available or likely to be developed in the immediate future for predicting the performance of coarse aggregate in portland cement concrete that is subjected to freezing and thawing exposure. A systematic approach to acceptance testing, taking advantage of a trained petrographer and using a battery of tests, seems to be indicated. There is also a need for an accumulation and evaluation of field performance under a variety of exposure conditions for correlation with test data on aggregate sources and individual fractions.

The accompanying figure indicates the test possibilities applicable to aggregate acceptance programs and how they might be used to meet varying objectives. A petrological examination of all sources is suggested as a first step. The left branch of the figure covers the large number of cases where general acceptance of a source is to be determined. If field performance information is available on aggregates with similar characteristics, a relative rating by two or more of the methods listed may be sufficient. The slow cooling (Powers) method may be used for cases where a period of frost resistance is desirable or no field experience is available for similar aggregates.

The right branch of the chart is appropriate for cases where economical aggregate sources in the "intermediate" field performance range must be evaluated. The sample is separated into relatively homogeneous fractions and the performance of each rated by single-particle tests and the slow cooling method when a determination of frost resistance is desired. The effect on performance of the deleterious fractions should be evaluated and beneficiation required if the source material is not considered suitable for the exposure anticipated.

It is hoped that this study of available and recently developed aggregate test methods wil result in a reduction of unnecessary testing and the saving of time and money in some instances where a rapid screening approach will be adequate for acceptance or rejection of an aggregate source. In addition, the more sophisticated method for predicting coarse aggregate performance in concrete under simulated field conditions should provide a basis for better utilization of marginal aggregate sources and improving the performance of highway pavements and structures under a variety of exposure conditions.



Procedural approaches to frost-susceptibility tests.

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ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 4-3(1) by Virginia Polytechnic Institute, with Richard D. Walker, Associate Professor of Civil Engineering, as Principal Investigator. Harry J. Pence, Research Assistant, was in charge of the concrete laboratory and supervised all of the laboratory work. Wei Jen Ong, Research Assistant (now an engineer with the West Virginia State Road Commission), was responsible for the porosimeter work reported in this final report. William H. Hazlett, Instructor of Geology, was responsible for all of the petrography work.

The facilities and equipment used were those of the Civil Engineering Department, Virginia Polytechnic Institute. The project was under the general administration of the Institute's Research Division.

ONE-CYCLE SLOW-FREEZE TEST FOR EVALUATING AGGREGATE PERFORMANCE IN FROZEN CONCRETE

CHAPTER ONE

INTRODUCTION AND RESEARCH APPROACH

The ability to determine what coarse aggregates, when used in concrete, are capable of resisting the forces of freezing and thawing has become increasingly important as sources of available aggregates of known satisfactory performance are being depleted in many areas of the United States. Conventional freezing-and-thawing tests have shown great promise in evaluating the performance of a concrete aggregate, at least for a particular geographical area. However, these tests are time consuming, generally requiring a minimum of one month to achieve definite results.

This investigation, which began in 1963, had for its purpose the development of a simple and fast test that could be used to identify aggregates that cause poor concrete performance when frozen. In the interim report (published as *NCHRP Report 12,* 1965) definite progress toward the goal of a quick and simple test was reported. This initial work consisted of placing 3 x 3 x 16-in. concrete specimens in a standard deep-freeze unit and making strain measurements with a Whittemore strain gauge every ten minutes over a 3- to 4-hr period. It was found that the slope of the change in length versus time curve correlated well with aggregate durability as defined by standard freezing-and-thawing tests.

However, the initial work, although extremely promising, used only a limited number of aggregates. This final report describes work toward refining the test method and its application to aggregates selected from many parts of the United States and representing a variety of rock types.

SCOPE

Nineteen different aggregates were used in concrete to fabricate a total of 336 3 x 3 x 16-in. specimens. Half of the specimens, after curing, were directly exposed to alternate cycles of freezing and thawing while in water in accordance with ASTM test designation C290-63T. The specimens were measured for length, weight, and dynamic modulus at the end of a specified number of cycles. Also, length-change measurements were made at intervals of approximately 2 F during most of the initial freezing-and-thawing cycle. The other half of the specimens, after curing, were exposed to a single slow-freeze cycle in a deep-freeze unit. Length-change and temperature measurements were made at 5- to 15-min intervals over a 4-hr period.

In the initial investigation, 14-day curing periods were used. Later work has shown that a 7-day curing period is adequate, and this was adopted for the investigation reported here.

From the data obtained, correlations were made between concrete durability as measured by the rapid freezing-and-thawing test in water (ASTM C290-63T) and certain characteristics of the temperature length-change and time length-change curves. All length-change measurements were made with a Whittemore strain gauge.

Aggregates Studied

Nineteen coarse aggregates were included in this study. Table 1 gives a very brief general description of these aggregate sources.

Aggregates A, C, and N were used in the initial study as reported in the interim report. These aggregates had designations of G, F, and A in that report. Several other aggregates, prepared by the Pennsylvania State University, were also tested. The physical characteristics of the aggregates are briefly described in Table 1.

Concretes Studied

Aggregates A, B, and C were used in concrete to determine the effect of curing time on test results. For these mixes, Aggregate N was blended with Aggregates A and C. Aggregate B was used alone. Aggregates D through S were used separately to accomplish the main objective of the study. All concrete mixes were designed using a low-alkali Type I cement, a cement factor of 5.5 bags per cu yd, an air content of 5.5 percent, and sufficient water to obtain a 3-in. slump. A single high-quality quartzite sand was used throughout the entire project. The sand was from the same aggregate source as coarse Aggregate N.

TABLE 1

PHYSICAL CHARACTERISTICS OF COARSE AGGREGATES

AGGR.	SPECIFIC GRAVITY BSSD	SPECIFIC GRAVITY BULK DRY	ABSORPTION (%)	EST. FIELD PERFORMANCE	BRIEF DESCRIPTION
A	2.39	2.23	7.41	Very poor	Ohio river chert gravel
B	2.73	2.68	2.05	Good	Glacial gravel northern Midwest
C	2.53	2.45	3.09	Poor	Midwestern chert river gravel
D	2.61	2.55	2.33	—	Eastern ridge and valley gravel
E	2.54	2.45	3.72	Poor	Eastern Appalachian river gravel
F	2.42	2.28	6.32	Poor	Interior low plateau river gravel
G	2.54	2.44	3.71	—	Volcanic material Western U.S.
H	2.66	2.60	2.39	Good	Midwestern glacial gravel
I	2.67	2.64	1.57	—	Northeastern U.S. crushed gravel
J	2.64	2.60	1.58	—	Eastern Piedmont crushed gravel
K	2.65	2.60	1.85	—	Central U.S. crushed limestone
L	2.66	2.63	1.02	—	Central U.S. crushed limestone
M	2.70	2.68	0.72	—	Central U.S. crushed limestone
N	2.63	2.61	0.52	Good	Eastern Piedmont quartzite gravel
O	2.70	2.67	0.95	—	Western igneous crushed stone
P	2.27	2.05	10.6	—	Western igneous crushed stone
Q	2.74	2.73	0.57	—	Southeastern Piedmont crushed stone
R	3.07	3.06	0.53	—	Eastern Piedmont crushed stone
S	2.64	2.59	1.97	—	Central U.S. crushed limestone

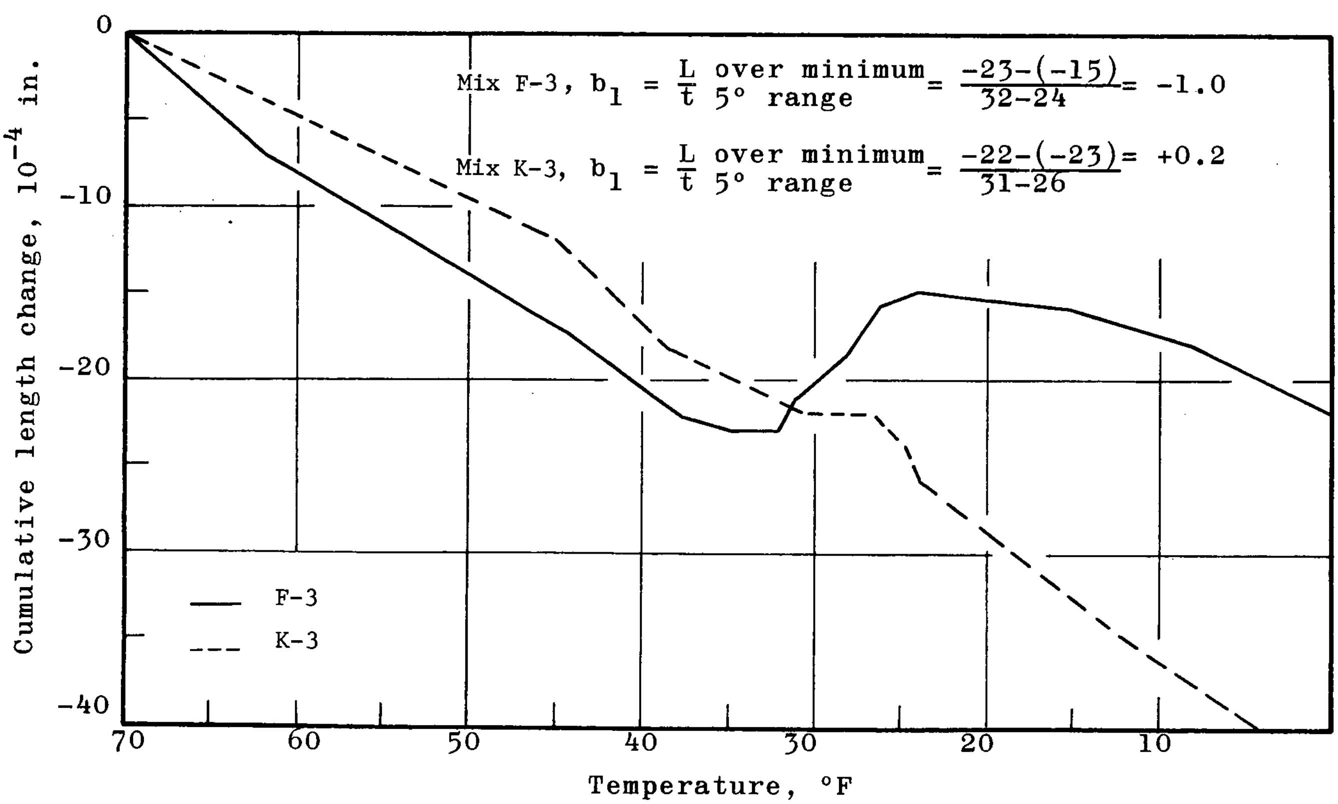

Figure 1. Example of calculation of b_1 temperature slope.

Mixing and Testing Procedures

The interim report should be referred to for general mixing and testing procedures. A detailed description of the one-cycle deep-freeze test is found in Chapter Four of this report. The coarse aggregates used were saturated using a vacuum of about 2 cm of mercury.

Before presenting the results, it is necessary to repeat the definition of several terms previously described in the interim report. DF_{100}, DF_{200}, and DF_{300} are the 100-, 200-, and 300-cycle durability factors as calculated by procedures described in ASTM designation C290-63T. These factors range from 0 to 100; for purposes of discussion in this study the 100-cycle durability factors have the following somewhat arbitrary meanings:

(1) Group I, $DF_{100} = 0\text{-}5$, indicative of potentially extremely poor field performance

(2) Group II, $DF_{100} = 6\text{-}30$, indicative of potentially poor field performance

(3) Group III, $DF_{100} = 31\text{-}80$, indicative of potentially fair field performance

(4) Group IV, $DF_{100} = 81\text{-}100$, indicative of potentially good field performance

Although these groupings are necessarily arbitrary, they agree quite well with the known field performance of many of the aggregates used in this study. A durability factor representing an average of DF_{100}, DF_{200}, and DF_{300} is presented along with 100-cycle factors. This average DF should give a better picture of the total performance of a specimen over a period of 300 cycles, but for all practical purposes it is felt that the DF_{100}, which in most cases requires less time to obtain, serves just as well as an indicator of concrete durability.

Another term to be defined is the minimum 5 F— temperature slope, b_1. This is the minimum slope that can be found within a range of 5 F or more, on the length change-temperature curve obtained during the first 3 hr of the first freeze of each specimen. Figure 1 illustrates the calculation of b_1.

A third term is the time slope, b_t, which is the minimum slope that can be found within a 20-min (⅓-hr) or greater range on the length-change vs time curve obtained during the first 3 hr of the first freeze of each specimen. Figure 2 shows an example of the calculation of b_t, the time slope.

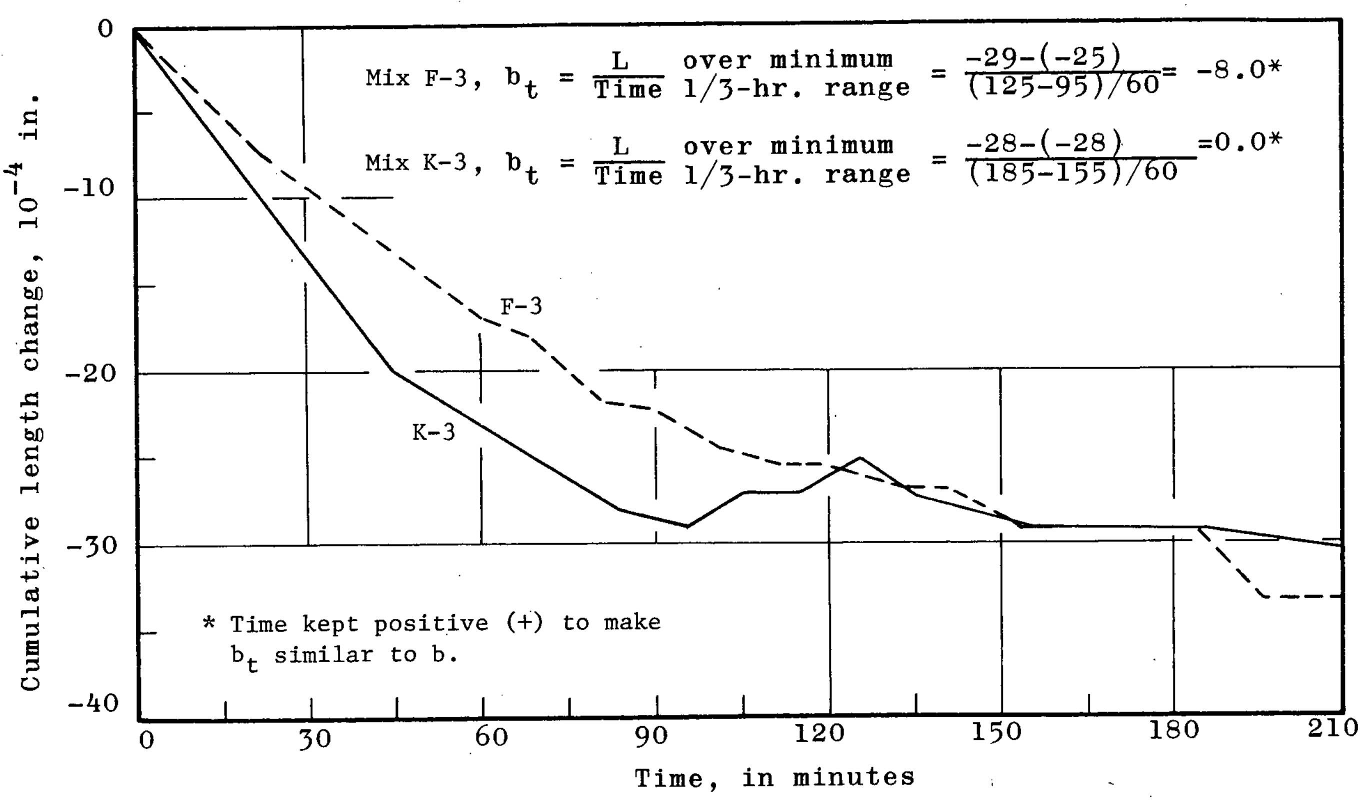

Figure 2. Example of calculation of b_t, time slope (Refer to interim report—NCHRP Report 12, p. 16.)

CHAPTER TWO

FINDINGS

The findings presented and discussed in the main body of the report deal only with the main objectives of this project; that is, the verification of the relationship between DF_{100} and b_t shown in the interim report and the detailing of the one-cycle slow-freeze test procedure.

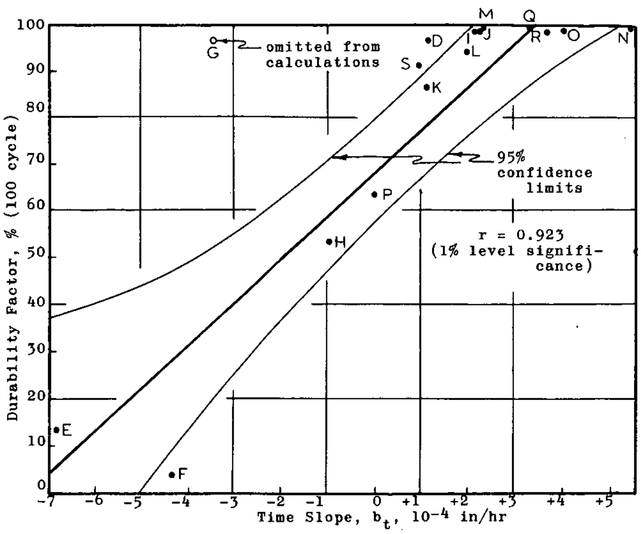

Figure 3. Relationship between DF_{100} and b_t for specimens exposed to slow-freeze test.

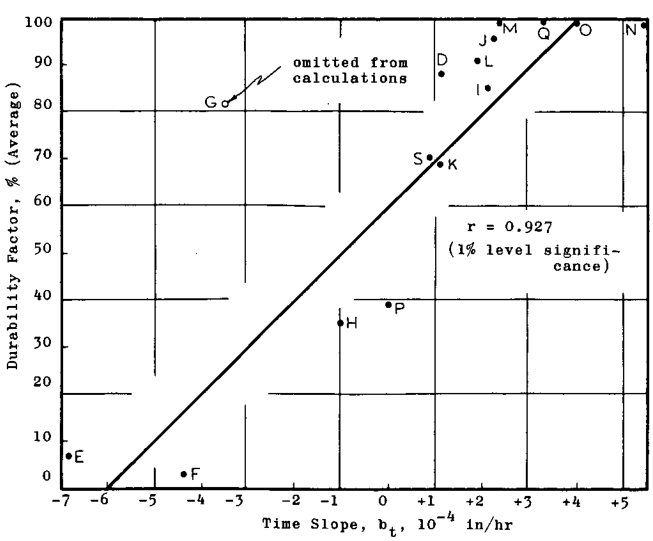

Figure 4. Relationship between average DF and b_t for specimens exposed to slow-freeze test.

RELATIONSHIPS BETWEEN DURABILITY FACTOR AND TIME AND TEMPERATURE SLOPES

The confidence limits shown in Figure 3 were calculated using the model

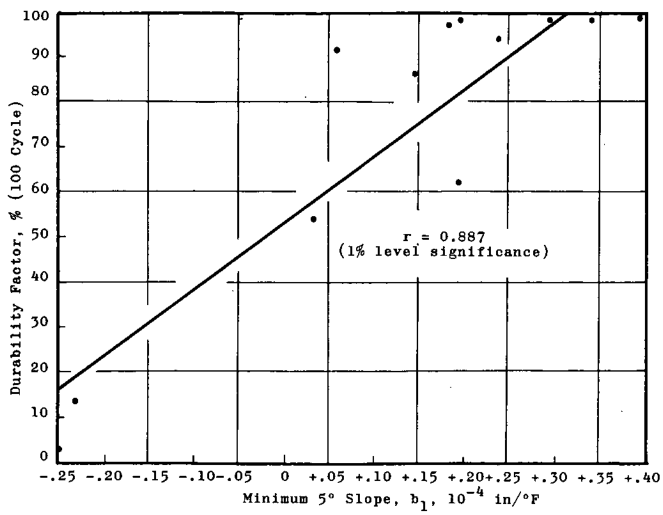

Figure 5. Relationship between DF_{100} and b_1 for specimens exposed to slow-freeze test.

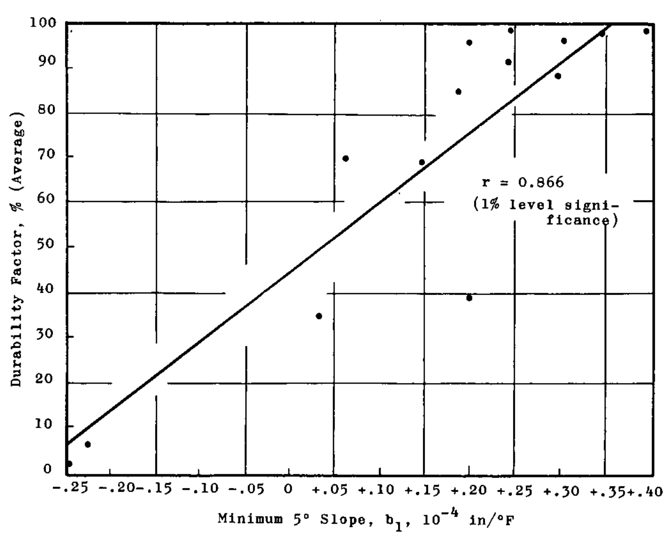

Figure 6. Relationship between DF and b_1 for specimens exposed to slow-freeze test.

$$yi \pm t_{1/2,\, N-2}\sqrt{MS_E}\sqrt{\frac{1}{N}+\frac{(Xi-\bar{X})^2}{S_{xx}}}$$

Reference should be made to standard statistical texts for this and other calculations such as correlation coefficients. Duncan * is such a text.

Figures 3 and 4 show the relationships between the durability factors DF_{100} and average DF and the time slope, b_t. Aggregates A, B, and C are not shown in these compilations as they were used in preliminary mixes that utilized somewhat different procedures and results of which are shown in Appendix A. By omitting Aggregate G from the calculations, a rather high correlation coefficient is obtained for both sets of data. Aggregate G is a porous volcanic rock which apparently is capable of giving large dilations without large losses of durability. With the exception of Aggregate G, all aggregates in concrete having 100-cycle durability factors of 80 or more, had temperature slopes (b_t's) of greater than +0.9. The very low durability concretes had temperature slopes of −4.4 and −6.8.

* Duncan, A. J., "Quality Control and Industrial Statistics." Richard D. Irwin, Inc., Homewood, Illinois.

Figures 5 and 6 show the relationship between the durability factors DF_{100} and average DF and the temperature slope, b_1. As shown in the interim report, a similar but not as strong a relationship exists between DF and b_1 as for DF and b_t.

OTHER MEASURES OF DURABILITY FROM SLOW-FREEZE TEST DATA

Temperature Durability Index (DI) and Time Durability Index (DI_t) are two possible measures of concrete durability that were also investigated. Figures 7 through 10 show the relationship of these values, with DF_{100} and average DF. The values of DI or DI_t were calculated by subtracting from b_1 or b_t the slope of the temperature or time vs length-change curve (b_n, b_{tn}) from the beginning of the test down to 40 F or about ¾ hr. Thus, DI and DI_t represent the deviation of the slope during freezing from the

TABLE 2

DATA SUMMARY, AVERAGES, AGGREGATES D THROUGH S

AGGR.	F&T [a] OR SF [b]	NO. SPECI-MENS	DF_{100} [c]	AVG [c] DF	b_t (10^{-4}IN/HR)	b_1 (10^{-4}IN/°F)	DI (10^{-4}IN/°F)	DI_t (10^{-4}IN/HR)
D	F&T	8	97	89 [d]	+0.5	+0.085	−0.391	—
	SF	8	97	89 [d]	+1.2	+0.296	−0.264	−20.4
E	F&T	8	13	8	−8.3	−0.496	−0.992	—
	SF	8.	13	8	−6.8	−0.227	−0.811	−31.6
F	F&T	8	3	1	−1.4	−0.092	−0.614	—
	SF	8	3	1	−4.3	−0.204	−0.878	−28.8
G	F&T	6	96	82 [d]	−0.2	+0.052	−0.371	—
	SF	6	96	82 [d]	−3.5	+0.018	−0.413	−19.2
H	F&T	8	54	35	−0.5	+0.017	−0.433	—
	SF	8	54	35	−1.0	+0.028	−0.517	−21.2
I	F&T	8	98	85 [d]	+2.7	+0.244	−0.304	—
	SF	8	98	85 [d]	+1.7	+0.179	−0.357	−21.4
J	F&T	8	98	96 [d]	+2.3	+0.119	−0.399	—
	SF	8	98	96 [d]	+2.2	+0.200	−0.406	−22.6
K	F&T	8	86	69 [d]	+0.9	+0.054	−0.264	—
	SF	8	86	69 [d]	+1.1	+0.149	−0.308	−16.4
L	F&T	4	94	92	−0.15	+0.092	−0.340	—
	SF	4	94	92	+2.1	+0.235	−0.252	−17.6
M	F&T	8	100	99 [d]	+3.0	+0.177	−0.254	—
	SF	8	100	99 [d]	+2.7	+0.244	−0.261	−18.5
N	F&T	8	100	100 [d]	+2.7	+0.315	−0.255	—
	SF	8	100	100 [d]	+5.8	+0.396	−0.297	−22.1
O	F&T	6	98	99 [d]	+3.6	+0.264	−0.163	—
	SF	6	98	99 [d]	+4.3	+0.353	−0.142	−17.4
P	F&T	4	64	39	−0.3	+0.072	−0.316	—
	SF	4	64	30	0	+0.244	−0.242	−17.1
Q	F&T	8	100	100 [d]	+2.2	+0.135	−0.350	—
	SF	8	100	100 [d]	+3.3	+0.244	−0.281	−18.4
R	F&T	8	98	97 [d]	+2.9	+0.196	−0.264	—
	SF	8	98	97 [d]	+3.6	+0.299	−0.214	−14.8
S	F&T	8	91	70 [d]	+1.0	+0.059	−0.382	—
	SF	8	91	70 [d]	+0.9	+0.98	−0.354	−15.6

[a] Specimen put directly into freezing and thawing (ASTM C290).
[b] Specimens initially placed in one-cycle slow-freeze test.
[c] Variability in DF calculations was sufficiently small to permit value to be based upon all specimens, F&T, and SF.
[d] Value estimated, a few specimens not having quite completed 300 cycles of freezing and thawing at time of report.

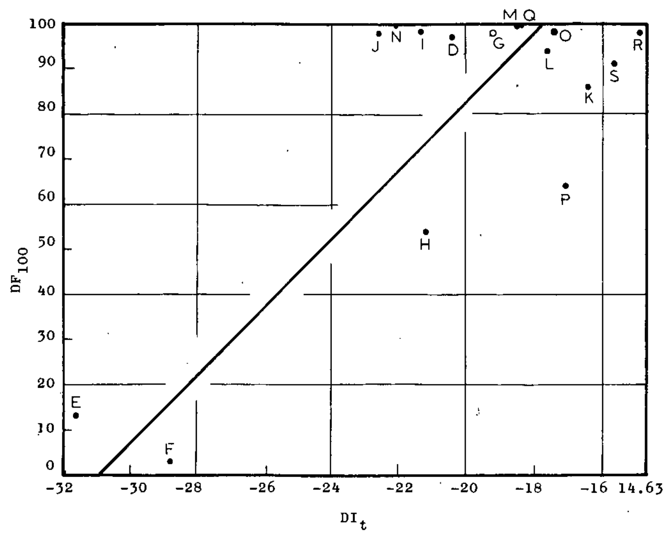

Figure 7. Relationship between DF and DI$_t$ for specimens exposed to slow-freeze test.

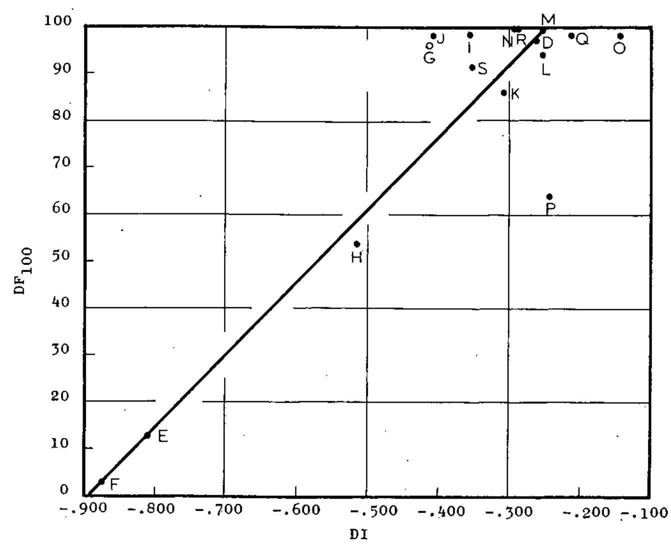

Figure 9. Relationship between DF$_{100}$ and DI for specimens exposed to slow-freeze test.

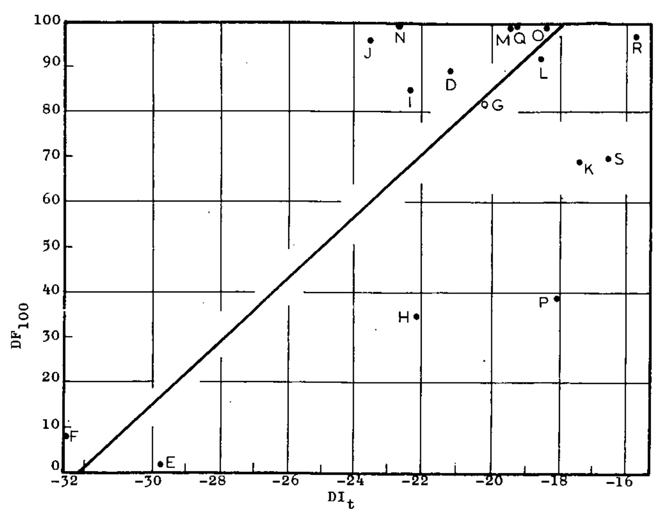

Figure 8. Relationship between average DF and DI$_t$ for specimens exposed to slow-freeze test.

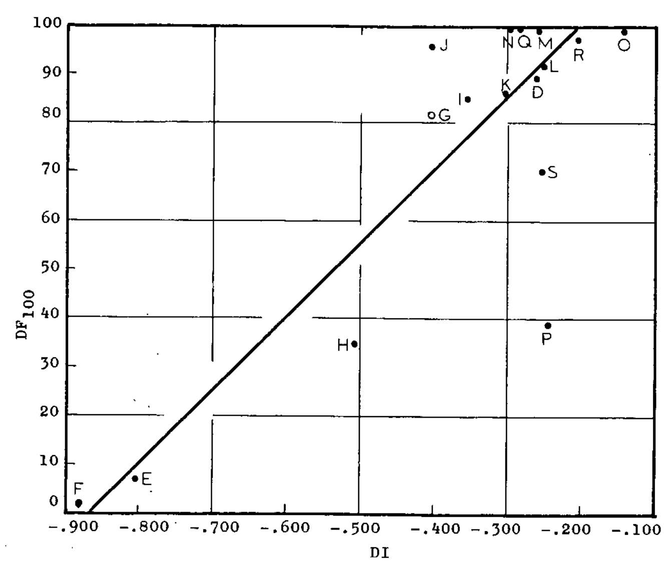

Figure 10. Relationship between average DF and DI for specimens exposed to slow-freeze test.

slope prior to freezing temperatures being reached. The idea is to remove the bias of the natural slope of a particular concrete.

The relationships obtained are not quite as satisfactory as with b_1 and b_t. However, it should be noted that for the one aggregate having unusually large negative b_1 and b_t values for its durability level, Aggregate G, the DI and DI$_t$ fall into their proper groupings. But, because of lack of distinction of middle durability aggregates, the DI and DI$_t$ values as calculated here are not as valuable as the b_1 and b_t values. If more accurate length-change measurement procedures were developed, the DI and DI$_t$ values could prove to be the better statistic.

CHAPTER THREE

INTERPRETATION AND APPLICATION

RELATIONSHIP BETWEEN DF_{100} AND B_t

In general the relationship between DF_{100} and b_t indicated in the interim report has been verified by the data presented here. The essential difference between the present data and the results shown in the interim report is that the b_t's of the present data are somewhat higher. It is felt that this is a result of the shorter curing time used (7 days as against 14 days). This is discussed in Appendix A.

Confidence limits are placed on the DF_{100} vs b_t plot for illustrative purposes only. These limits serve to indicate the relative significance of the data obtained to date. If more data were obtained, it is thought that the limits shown could be considerably narrowed. With the results as shown, it is possible to enter Figure 3 with a value of b_t, of for example, zero, and predict a range of durability factors, in this case from 58 to 80 at the 95 percent confidence level.

Knowledge of durability factor is most valuable when correlations have been made between it and field performance for a specific use under specific climatic conditions. Subcommittee III-e of ASTM Committee C-9 on Concrete and Concrete Aggregates revision of ASTM C-33, Specification for Concrete Aggregates, will probably make the limiting values of specific tests dependent upon use and climatic area. Progress along this line should make the normal freezing-and-thawing tests, or results of such tests as predicted by the one-cycle slow-freeze method, of much greater value.

With the limited results presented here, it should be possible to use the one-cycle slow-freeze test as a screening device. In no case has an aggregate exhibiting a b_t of +1.0 or greater had a DF_{100} of less than 80 percent. Also, it should be noted that the majority of the data points are in the $DF_{100} = 80+$ present range. Thus, initially it should be possible to say that concretes having a b_t greater than +1.0 are satisfactory and all others should undergo routine freeze-thaw testing. At the other end of the spectrum, it might be said that if b_t is less than —4.0, the aggregate is definitely very poor. The data in this area are scarce but definite in that if b_t's are this low, durability factors are low. In the interim report, two concretes with low durability were reported with somewhat higher b_t's but this would mean, under this recommendation, that these would undergo routine freeze-thaw testing. In no case has a high durability factor concrete slipped into this low range of b_t. The nearest exception specifically would be Aggregate G, which was a high absorptive volcanic aggregate with a vesicular pore structure. If such aggregates could be screened petrographically, the limit of b_t could be put closer to —2.0 than —4.0.

RELATIONSHIP BETWEEN DURABILITY FACTOR AND TEMPERATURE SLOPE

The relationships between durability factor and temperature slope b_1 are presented to show that the time relationship is at least as significant, if not more so. The one-cycle slow-freeze test is a simpler, less expensive test to conduct with the time relationship than if temperature measurements were required.

It must be kept in mind that these tests have been run using vacuum saturated aggregates, since the results of the slow freeze test are obtained during the first freeze and sufficient time would not otherwise have elapsed for the aggregate to have reached a high moisture content. Any aggregate under good drainage conditions can perform well. Under poor drainage conditions so often prevalent, poorly performing aggregates gain moisture approaching that obtained under vacuum saturation.

CHAPTER FOUR

CONCLUSIONS AND RECOMMENDATIONS

The results of this and the interim report indicate that the one-cycle slow-freeze test could now be used as a screening test if durability factor is considered as a measure of potential field performance. The test has shown sufficient potential to warrant testing in other laboratories to substantiate the findings reported here.

Until more definitive information is developed, the following conclusions can be made from the results:

(1) $b_t \geqq +1.0$, assume DF_{100} is greater than 80 percent and no further testing required.
(2) $b_t \leqq -4.0$, assume DF_{100} is less than 50 percent and durability is definitely questionable under conditions of freezing and thawing in the presence of moisture.
(3) $-4.0 \leqq b_t \leqq +1.0$, ASTM C290 should be run to evaluate durability factor.

The significance of durability factor in relation to field performance must be determined for a specific aggregate use and geographic location. It is recommended that the following procedure be considered by Subcommittee III-e of ASTM Committee C-9 on Concrete and Concrete Aggregates.

ONE-CYCLE SLOW-FREEZE TEST METHOD (PROPOSED)

The following description of the slow-freeze test is provided as a guide for those laboratories which may wish to evaluate it.

Scope

This method covers the estimation of the resistance of concrete specimens to rapidly repeated cycles of freezing and thawing in water in the laboratory (as described in ASTM C290-63T). It is intended as a screening test to separate concretes having middle range durabilities near the extremes. ASTM C290-63T, *Resistance of Concrete Specimens to Rapid Freezing and Thawing in Water*, is recommended for use on the concretes falling in the middle durability range. It is anticipated that this procedure will be used as part of a series of tests for evaluating coarse aggregates to be used in concrete exposed to freezing and thawing.

Apparatus

The freezing apparatus shall consist of a chamber that will maintain air temperatures of OF ± 3F. Apparatus similar to the common home chest-type food freezer will generally be satisfactory.

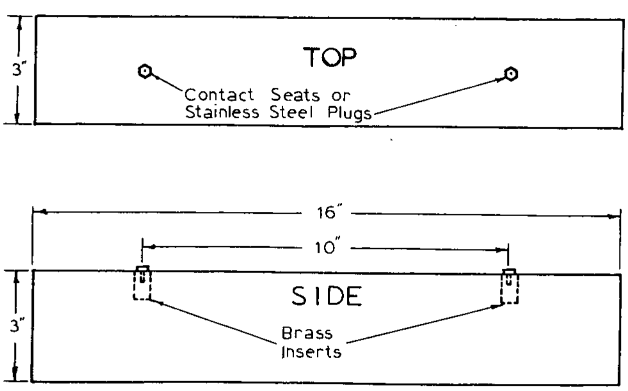

Figure 11. Strain plug installation.

A wire basket or shelf, sufficiently large and stable to hold either 3 x 3 x 16-in. or 3 x 4 x 16-in. specimens, shall be affixed in the upper part of the chamber.

Strain Gauge Equipment

A Whittemore strain gauge is recommended for measuring changes in length of the specimen. This gauge reads to 0.0001 in. and when used in conjunction with properly installed stainless-steel strain plugs, will be accurate to ±0.00005 in. or better.

Test Specimens

The specimens for use in this test shall be prisms made and cured in accordance with the applicable requirements of the *Method of Making and Curing Concrete Compression and Flexure Test Specimens in the Laboratory* (ASTM Designation C192). Specimens 3 x 3 x 16 in. or 3 x 4 x 16 in. should be used if the suggested time limits given under procedures are to be followed.

Coarse aggregates should be vacuum saturated before incorporation into the concrete to ensure an adequate moisture content at the time of test.

Stainless-steel strain plugs should be installed as shown in Figure 11. The brass inserts which carry the stainless-steel plugs are expendable and may be installed in the fresh concrete by having them screwed at 10-in. centers to a strip of steel. Thus, the inserts can be vibrated into the concrete while maintaining proper spacing and remaining level. Stainless-steel plugs are required to maintain the necessary degree of accuracy in the length-change measurement.

For this test the specimens shall be stored in saturated limewater from the time of their removal from the molds (specimens should be in the molds for 24 hr) until the freezing tests are started. This normally will be a minimum of six days. All specimens to be compared with each other shall be of the same initial dimensions.

Procedure

Immediately after the specified curing period, the specimens shall be tested for fundamental transverse frequency and weighed in accordance with the *Method of Test for Fundamental Transverse, Longitudinal, and Torsional Frequencies of Concrete Specimens* (ASTM Designation C215). The specimen should be measured for length to the nearest 0.0001 in. The specimens shall be protected against loss of moisture between the time of removal from curing and start of the freezing test.

Freezing tests shall be started by placing the specimens horizontally in the apparatus and then noting the time to the nearest minute. Length-change measurements shall be made at 15-min intervals until the total elapsed time is 45 min. Subsequent measurements shall be made at 5-min intervals until 3 hr have elapsed. The specimens are then removed from the freezing chamber and placed in limewater at 70 F for approximately 24 hr, at which time the specimen is again tested for fundamental transverse frequency, weighed and measured for length. If desired, the specimens can then be subjected to ASTM Designation C290, *Resistance of Concrete to Rapid Freezing and Thawing.*

Calculations

Time slope, b_t — numerical values of time slope shall be calculated as follows:

$$b_t = \frac{\Delta L}{\Delta \, Time} \qquad (1)$$

which is calculated to represent the minimum slope that can be found within a 20-min (⅓ hr) or greater range up to 1 hr on a length-change time curve. Δ *Time* is always recorded as a positive (+) number. Minimum slope is the lowest value; thus a —2.0 slope is lower than a +0.1 slope. The algebraic sign of ΔL is determined by subtracting the second length-change value from the first as they occur on the time scale. Units of the calculation shall be in 10^{-4} in./hr.

Report

Because of the variability inherent in specimen-to-specimen test results, it is recommended that b_t values used be the average of results of eight specimens. Until more definitive information is developed, the following conclusions can be made from the results:

(1) $b_t \geqq +1.0$, assume 100 cycle Durability Factor (DF_{100}) is greater than 80 percent and no further testing required.
(2) $b_t \leqq -4.0$, assume DF_{100} is less than 50 percent and durability is definitely questionable under conditions of freezing and thawing in the presence of moisture.
(3) b_t between —4.0 and +1.0, ASTM C290 should be run to evaluate durability factor.

The significance of durability factor in relation to field performance must be determined for a specific aggregate use and geographic location.

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APPENDIX A

CURING TIME STUDIES

The data given in this appendix represent the preliminary work accomplished before beginning the work described in the main body of the report. Three basic mix designs using: (1) combination of Aggregates A and N, (2) Aggregate B, and (3) combination of Aggregates C and N, were employed.

Certain specimens in this study underwent the slow-freeze test while immersed in water. For all of these preliminary tests a fan was used in the freezing compartment to reduce freezing time. The fan was dispensed with in the main body of tests and the freezing-in-water technique did not prove to be practical. Table A-1 represents the data obtained from the one-cycle slow-freeze specimens that were frozen in air.

Looking at the low durability concrete using Aggregates A and N, it was seen that with 3-day curing and Type I cement, first freeze dilations were not occurring as they were with the other treatments. It is felt that expansion of the aggregate was not resisted by the low-strength mortar, causing local failure of the mortar immediately surrounding the particle leaving the remaining mortar free to contract. Thus, over-all expansion did not occur. When 3-day curing is used in connection with high early strength cement, dilations are greatly increased. Seven- and 14-day curing produced concrete that showed progressively larger dilations which could be a result of both a stronger mortar strength and higher moisture content of the concrete.

Since the objective of the project was to reduce the test time to a minimum, it was decided to use a 7-day curing period. The 3-day curing period was unsatisfactory, and it was thought that the 7-day curing period would give sufficiently high dilations to make a relationship between durability and b_1 and b_t possible. The main disadvantage in using the 7-day curing period is that direct comparison could not be made with the 14-day results shown in the interim report. It is believed that this disadvantage was outweighed by shortening the total test period from 14 to 7 days.

Additional research is needed to investigate other techniques of adequately shortening curing time. Continued study is progressing at Virginia Polytechnic Institute in this area, and in particular using heated curing water.

DETERMINATION OF LENGTH OF SLOW-FREEZE CYCLE

The suggested length of the slow-freeze cycle is somewhat arbitrary but not without foundation. The 3-hr time limit on the test prevents any of the specimens tested from approaching 0 F. Table A-2 illustrates this. The specimens were frozen while resting in a wire rack in a 20-cu ft deep-freeze unit. No fan was used, so the specimens were in a still air environment of about 0 F to —5 F. As the temperature of the specimen gets closer to 0 F, more time is required per °F drop in temperature. Thus, if a time limit were not placed on the test, a specimen of low moisture content and rapid cooling properties would have a flatter time slope, b_t, when it approached 0 F.

The minimum time slope, b_t, probably occurs when the most water is freezing in the specimen and this occurs at all different times depending upon the aggregate used. By restricting the test to 3 hr, most of the specimens (except those with very high moisture content) will pass 20 F and approach 10 F and the b_t will be the result of freezing water rather than the specimen's inability to get past 0 F.

SUPPORTING DATA

Table A-1 gives the average data for the curing time study (Aggregates N-N, B, and C-N) and are not repeated here. Table 2 (Chapter 2) summarizes the remaining data.

TABLE A-1

CURING TIME STUDY DATA

AGGR.	AVG. DF_{100}	AVG. b_1 (10^{-4} IN./°F)	AVG. b_t (10^{-4} IN./HR)	CURING TIME (DAYS)	CEMENT TYPE	NO. SPECIMENS TESTED
A–N	3	+0.200	−1.2	3	I	6
A–N	3	−1.319	−14.8	3	III	6
A–N	2	−3.477	−21.8	7	I	6
A–N	2	−6.430	−31.4	14	I	6
B	95	+0.050	−0.2	7	I	4
B	88	+0.038	0.0	14	I	4
C–N	71	+0.425	+3.0	7	I	4
C–N	45	+0.183	−0.8	14	I	4

Individual specimen data are quite voluminous and are on file with the Research Agency, Virginia Polytechnic Institute.

Included in Table 2 are b_t, b_1, and DI values for specimens exposed only to ASTM C290, the alternate freezing-and-thawing test. These values are based on the first cycle of freezing and are presented only as information since they are not discussed in the report. Table 1 (Chapter 1) summarizes the physical characteristics of the aggregates used in the study.

TABLE A-2

TIME FOR SPECIMENS TO REACH SPECIFIED TEMPERATURES

	70 F-20 F		70 F-10 F		70 F-0 F	
AGGR.	(MIN.)	(MAX.)	(MIN.)	(MAX.)	(MIN.)	(MAX.)
D	2:10	2:30	3:05	3:35	4:45	5:15
E	2:20	2:45	3:15	3:50	5:20	8:00
F	2:35	3:15	3:50	5:10	6:10	7:40
G	2:15	2:50	3:40	4:45	5:10	6:15
H	1:55	2:55	2:40	4:30	4:10	6:10
I	2:00	2:15	3:00	3:30	4:10	6:25
J	1:45	2:35	2:40	4:00	4:10	7:50
K	1:55	2:30	3:05	3:55	5:05	5:45
L	2:15	2:40	3:30	3:55	6:25	9:00
M	1:40	2:25	2:55	3:50	4:30	6:20
N	1:40	2:00	2:40	3:15	4:10	6:25
O	1:40	2:35	2:30	3:55	3:45	6:55
P	3:05	3:25	4:25	4:35	6:40	7:10
Q	1:25	2:25	2:20	3:40	3:25	5:45
R	1:45	2:10	2:45	3:30	4:15	5:55
S	2:20	2:45	3:30	4:10	5:05	7:30
Final	1:25	3:25	2:20	5:10	3:25	9:00

APPENDIX B

PETROGRAPHIC STUDIES OF AGGREGATES

COMPOSITION OF AGGREGATES

The composition of aggregates is dealt with in some detail in this appendix:

Aggregate A.—Refer to Aggregate G in the interim report (*NCHRP Report 12*).

Aggregate B.—Refer to Table B-1.

Aggregate C.—Refer to Aggregate F in the interim report.

Aggregates D and E.—Refer to Tables B-2 and B-3.

Aggregate F.—Approximately 98 percent of Aggregate F is composed of chert, most of which is limonite-stained and at least moderately weathered. The remaining aggregate particles include quartz, sandstone, slag, and mollusk shells. The cherts are white, grey, buff, tan, brown, and yellow on freshly-fractured surfaces but most possess a brown or yellow-brown, limonite-stained weathered rim. The exterior surfaces of many particles are rough, irregular, and commonly pitted; others are relatively smooth.

Most of the cherts are composed predominantly of cryptocrystalline quartz. Chalcedony and fine-to-medium-grained anhedral quartz occur most commonly as minor constituents in approximately 15 percent of the particles.

Most cherts appear to be silicified limestones or dolomites and possess the textures and framework constituents characteristic of carbonate rocks. Approximately 60 to 70 percent contain fossil material and pellets. The fossils, although generally replaced by microcrystalline silica, are in some instances replaced by coarse-grained quartz or by chalcedony. Many of the pellet- and fossil-bearing cherts contain small quantities of silt and sand-sized detrital quartz.

A few chert particles contain up to 3 or 4 percent small (ca. 0.05 mm) rhombohedral dolomite crystals. Approximately 40 percent contain dolomolds, resulting from the dissolution of anhedral dolomite. These cavities average 0.05 mm in longest dimension and generally account for 1 to 3 percent of the volume of individual chert particles.

Forty to 60 percent of the cherts are fractured. In many instances, fractures developed during testing result from the reopening or extension of fractures originally present in the aggregate of interest. Most of the visible pores in the cherts are dolomolds or areas formerly occupied by fossil material.

Aggregate G.—Aggregate G contains approximately 75 to 80 percent volcanic rock of intermediate to basic composition (andesite to basalt), 15 to 20 percent limestones and dolomites, and relatively small amounts of chert, quartzite, and sandstone. The igneous particles are medium to dark grey, black, tan, brown, and red. Approximately 40 percent of the volcanics are coated or partially coated with buff- or tan-colored calcareous material. This limestone-like material is commonly laminated and may include extraneous, silt- and sand-sized debris.

TABLE B-1

PETROGRAPHIC ANALYSIS OF AGGREGATE B

CONSTITUENTS	PERCENT BY WEIGHT [a] 1-¾ IN.	¾-½ IN.	½-¼ IN.
Granite	2.3	1.9	1.0
Rhyolite	—	0.4	0.3
Basic igneous	1.9	0.5	0.8
Highly weathered basic igneous	1.1	0.8	0.6
Greenstone	0.6	0.9	1.9
Miscellaneous metamorphics	1.9	0.9	0.7
Quartzite and quartz	1.0	2.3	0.7
Sandstone (largely graywacke)	4.9	3.4	4.5
Limestone	1.1	2.1	1.3
Dolomite	67.6	63.3	65.2
Cherty dolomite	4.2	7.7	6.5
Chert	9.3	10.2	6.8
Dolomitic and calcareous chert	4.1	5.6	9.7

[a] Based on analysis of 11.2 lb of 1-¾ in., 4.1 lb of ¾-½ in., and 1.1 lb of ½-¼ in. aggregate.

The coatings on many particles are quite heavy and attain a maximum thickness of 1 mm or more.

Many of the volcanic rocks are vesicular and most are porphyritic. Minerals present as phenocrysts include plagioclase, olivine, clinopyroxene, hornblende, and biotite.

TABLE B-3

PETROGRAPHIC ANALYSIS OF AGGREGATE E

CONSTITUENT	PERCENT BY WEIGHT [a] 1-¾ IN.	¾-½ IN.	½-¼ IN.
Granite	2.1	3.6	3.3
Rhyolite and aplite	2.9	1.2	1.5
Basic igneous	0.7	0.4	0.8
Gneiss	2.9	3.2	3.2
Amphibolite and greenstone	1.6	0.8	1.2
Miscellaneous metamorphics	0.5	0.4	0.3
Highly weathered granite and gneiss	1.8	1.4	1.1
Highly weathered unidentified igneous and metamorphic	3.1	1.4	0.7
Quartzite and quartz	5.5	6.0	6.3
Sandstone	39.4	36.2	37.3
Siltstone	17.5	18.0	13.5
Shale	—	0.4	0.2
Limestone	3.9	3.6	2.1
Dolomite	1.4	2.4	0.8
Calcareous dolomite and dolomitic limestone	1.1	1.0	0.6
Cherty limestone and dolomite	1.6	1.3	1.6
Chert	13.2	18.4	23.5
Iron and manganese oxide particles	0.8	0.3	1.7
Cinders and coal	—	—	0.3

[a] Based on analysis of 11.6 lb of 1-¾ in., 4.3 lb of ¾-½ in., and 1.8 lb of ½-¼ in. aggregate.

TABLE B-2

PETROGRAPHIC ANALYSIS OF AGGREGATE D

CONSTITUENT	PERCENT BY WEIGHT [a] 1-¾ IN.	¾-½ IN.	½-¼ IN.
Granite	0.3	0.4	1.9
Gneiss	0.8	1.0	2.8
Miscellaneous metamorphics	0.7	0.3	0.9
Quartzite and quartz	18.6	18.5	11.5
Conglomerate	0.7	0.3	0.5
Sandstone	51.2	54.5	48.5
Siltstone	17.9	17.6	26.9
Shale	5.8	2.3	2.3
Chert	4.0	5.1	4.7

[a] Based on analysis of 10.3 lb of 1-¾ in., 4.0 lb of ¾-½ in., and 1.5 lb of ½-¼ in. aggregate.

These are embedded in a fine-grained and, commonly, glass-rich matrix characterized by high proportions of flow-oriented plagioclase microlites.

The carbonates, both limestone and dolomite, are generally buff or tan in color. The limestones are microcrystalline to medium-grained, while the dolomites tend to be fine-grained. Both may contain pellets, fossils, or silt- and sand-sized detrital quartz. They are little weathered to moderately weathered and are moderately absorbent.

The chert is grey, white, and buff in color and generally is not extensively weathered. However, a few of the buff-colored, dolomitic cherts are highly weathered, soft, and very absorbent.

Aggregates H, I, and J.—Refer to Tables B-4, B-5, and B-6.

Aggregate K.—Aggregate K consists of mottled blue and

TABLE B-4

PETROGRAPHIC ANALYSIS OF AGGREGATE H

CONSTITUENTS	PERCENT BY WEIGHT [a] 1-¾ IN.	¾-½ IN.	½-¼ IN.
Granite	3.9	2.2	1.5
Rhyolite and aplite	0.7	0.7	1.4
Intermediate and basic igneous	1.1	0.6	1.3
Gneiss	1.9	1.5	1.2
Greenstone and amphibolite	1.0	0.4	—
Miscellaneous metamorphics	2.2	1.7	1.7
Quartzite and quartz	0.8	2.7	3.3
Sandstone	6.1	5.4	4.6
Siltstone	0.3	—	—
Limestone	1.8	2.5	1.3
Cherty limestone	1.4	1.0	0.6
Dolomite	40.4	36.6	34.3
Cherty dolomite	13.0	9.0	13.4
Chert	17.4	26.3	27.4
Dolomitic and calcareous chert	8.0	9.4	8.0

[a] Based on analysis of 10.4 lb of 1-¾ in., 4.0 lb of ¾-½ in., and 1.3 lb of ½-¼ in. aggregate.

TABLE B-5

PETROGRAPHIC ANALYSIS OF AGGREGATE I

CONSTITUENT	PERCENT BY WEIGHT [a] 1-¾ IN.	¾-½ IN.	½-¼ IN.
Granite	0.5	1.8	2.3
Intermediate and basic igneous	—	1.0	0.4
Gneiss	1.5	0.7	1.7
Miscellaneous metamorphics	2.3	0.9	1.6
Quartzite	4.0	3.5	3.2
Sandstone	21.7	20.6	20.7
Siltstone	0.3	1.8	0.6
Shale	0.3	0.7	3.8
Limestone and dolomitic limestone	36.9	33.8	39.5
Dolomite and calcareous dolomite	27.6	28.8	19.5
Silty and sandy (arenaceous) limestone and dolomite	2.9	2.1	1.7
Cherty limestone and dolomite	0.9	1.2	1.1
Highly weathered limestone and dolomite	0.7	1.6	1.8
Chert	0.4	1.5	2.1

[a] Based on analysis of 10.8 lb of 1-¾ in., 4.4 lb of ¾-½ in., and 1.1 lb of ½-¼ in. aggregate.

TABLE B-6

PETROGRAPHIC ANALYSIS OF AGGREGATE J

CONSTITUENTS	PERCENT BY WEIGHT [a] 1-¾ IN.	¾-½ IN.	½-¼ IN.
Granite, granodiorite, syenite, and diorite	13.8	10.6	18.1
Gabbro, diabase, and peridotite	6.4	6.2	3.4
Gneiss and granulite	38.7	42.0	40.6
Amphibolite	4.1	2.4	2.0
Greenstone	2.1	1.2	0.7
Quartzite	11.2	13.5	15.1
Miscellaneous metamorphics	3.1	4.1	3.5
Conglomerate and conglomeratic sandstone	1.6	0.6	0.2
Sandstone	10.8	12.0	9.5
Siltstone	3.1	2.4	3.1
Shale and argillite	2.9	3.8	2.2
Chert	2.2	1.2	1.6

[a] Based on analysis of 9.7 lb of 1-¾ in., 4.5 lb of ¾-½ in., and 1.3 lb of ½-¼ in. aggregate.

gray and blue and tan, predominantly fine-grained, fossiliferous limestone. The particles are unfractured, unweathered or very little weathered, and nonabsorbent.

The limestone particles are composed almost entirely of calcite, which, in most particles, occurs as three textural components: (1) turbid microcrystalline calcite, (2) clear, fine- to coarse-grained calcite, and (3) calcareous fossils and/or pellets of microcrystalline calcite. However, the lime mud matrix and, to a lesser extent, the fossils and pellets have been at least partially replaced by granular mosaics of clear, coarser-grained calcite.

The microcrystalline calcite-sparry calcite ratio averages approximately 2:1. A few particles are composed almost entirely of microcrystalline carbonate, while others contain as little as 30 percent of this component. The content of fossils and pellets varies markedly but is generally in the 5 to 50 percent range.

The limestone particles are not fractured, but a few contain very fine, irregular stylolitic seams marked by minor concentrations of dark organic residue and iron oxide staining. No pores are visible microscopically.

Aggregate L.—Aggregate L is composed of unweathered, tan, fossiliferous, microcrystalline limestone. Many particles possess irregular stylolitic seams composed of dark brown or grey organic and argillaceous material and possibly finely disseminated iron oxide. Since these stylolites are zones of weakness along which rock tends to break, many particles contain partial coatings of the dark residue. Most of the limestone consists basically of dark, shelly fossil debris sparsely distributed and embedded in a matrix of microcrystalline or, less commonly, finely crystalline calcite. Some particles contain small masses of medium- and coarse-grained calcite occurring as a replacement of the microcrystalline carbonate or fossil material.

Aggregate M.—Approximately 95 percent of Aggregate M is composed of unweathered, grey and tan-grey, predominantly microcrystalline to finely crystalline limestone. A few of the particles are laminated, but many more contain very fine, dark, stylolitic seams. Although basically fine-grained, many of the limestones contain irregularly shaped masses of coarser-grained calcite. Fossils, oolites, and pelletiferous bodies occur in many aggregate particles, but the content of these framework constituents is seldom significant. Approximately 5 percent of the aggregate consists of moderately to highly weathered, buff-colored, calcareous dolomite. A few of these particles are extremely soft and appear to be somewhat cherty. Dark grey calcareous shale occurs only in trace proportions.

Aggregate N.—Refer to Aggregate A of interim report.

Aggregate O.—Aggregate O is composed predominantly of little weathered, medium- to coarse-grained diorite, quartz diorite, and granodiorite. The particles are unfractured, nonabsorbent, and contain no visible pores. These igneous rocks contain an average of 30 percent dark mafic minerals, chiefly hornblende and biotite, while the remainder is composed of the light-colored constituents, plagioclase, potassium feldspar, and quartz. Approximately 5 percent of the aggregate particles contain less than 5 percent of the ferromagnesian minerals and are very light in appearance. Three percent of the aggregate consists of dark grey or black, fine-grained diorite, gneiss, and biotite schist, while approximately 3 percent of the particles are of granitic composition.

Aggregate P.—Most particles in Aggregate P consist of moderately weathered, vesicular, porphyritic, apanitic igneous rocks. The presence of hornblende and, less commonly, plagioclase phenocrysts suggests an andesitic composition. These particles are very light to medium grey, tan, and pink in color. A few have bright orange or orange-red iron oxide staining. Approximately 8 percent of the aggregate particles are red in color. Some of these may be of rhyolite composition, but most appear to have derived their color from the oxidation of iron contained in the mafic minerals. Most of the particles are at least moderately absorbent as a result of the relatively high percentage of vesicles. The lighter-colored particles are earthy and cindery (scoriaceous) in appearance and tend to have larger and a higher percentage of vesicles compared with the darker particles.

Aggregate Q.—Approximately 99 percent of Aggregate Q consists of unweathered, medium to dark greyish-green and black, fine- to medium-grained gneiss, metagreywacke, amphibolite, and schist. Approximately 1 percent of the aggregate particles are composed entirely of vein quartz, while others contain quartz veinlets or portions of veins. Nearly all particles are tough, nonabsorbent, and contain no identifiable pores. The principal mineral constituents of the metamorphics include quartz, plagioclase, amphibole, mica (chiefly muscovite), and chlorite. A few of the particles contain layers or small, irregular masses of pink and white granitic material. Many of the rock particles possess a relatively indistinct metamorphic fabric. However, the mica- and chlorite-bearing schists, constituting approximately 5 percent of the aggregate, have a relatively well-defined schistosity as a result of the abundance, and preferred planar orientation, of the micaceous minerals.

Aggregate R.—Approximately 99 percent of Aggregate R is composed of black or green-black, medium- to coarse-grained, hornblende schist and amphibolite. Vein quartz and chert compose the remaining aggregate particles. The crushed metamorphic aggregate is unweathered or very slightly weathered, unfractured, and nonabsorbent. The basic mineral constituents are hornblende and plagioclase, while the accessory minerals include epidote, chlorite, garnet, and pyrite. A few particles contain calcite veinlets. The amphibolites contain approximately equal proportions of amphibole and plagioclase and are generally medium-grained, equigranular, and non-foliated. The schists contain a much higher percentage of hornblende. Some are almost monomineralic. They tend to be coarser-grained than the amphibolites and commonly possess a well-developed foliation and lineation as a result of the preferred orientation of the lath- or needle-shaped hornblende crystals.

Aggregate S.—Most particles of Aggregate S consist of unweathered, light grey to tan, fossiliferous, microcrystalline to finely granular limestone. Many particles contain dark stylolites composed of organic residue and argillaceous material. Dark grey partial coatings on the surfaces of some limestone particles result from fracture along the relatively weak stylolitic seams. Approximately 5 to 7 percent of the particles in each size fraction contain either numerous, very fine, argillaceous laminae or appreciable amounts of disseminated clayey material. The later argillaceous particles are commonly soft and grade into medium grey calcareous shale. The shale, however, is present only in trace proportions. Some of the limestones are oolitic or contain thin, oolite-bearing lamellae. Approximately 1 percent of the aggregate particles are moderately porous as a result of the differential solution of oolites or oolite cores.

SIGNIFICANCE OF PORE CHARACTERISTICS OF CONCRETE AGGREGATES

Verbeck (*1*) stated that aggregate constitutes about 75 percent of the volume of concrete. Consequently, the properties of the concrete are significantly influenced by the characteristics of the aggregates. As early as 1946, a significant statement by Rhoades and Mielenz (*2*) asserted that pore characteristics of aggregates are important controls of chemical and physical stability and they strongly influence the bond with cement. They significantly affect the strength of any material, and also determine absorption and permeability. As a result, they control durability under freezing-and-thawing conditions, and rate of chemical alteration.

It seems that pore characteristics are the dominant factor in determining the durability of aggregates.

Lewis, Dolch, and Woods (*3*) stated in this connection that it would be difficult to prove that any other physical property is of greater importance than the porosity characteristics (amount, size, and continuity of the pores) in either natural or artificial aggregates. Blanks (*4*), and Rhoades and Mielenz (*5*) showed that pore characteristics have an effect on the physical properties of a material. Larger void volumes mean a reduced specific gravity and higher porosity results in lower strength. Rhoades and Mielenz (*2*) found that both roughness and the pore characteristics of the surface zone affect the surface texture and the quality of bond between an aggregate particle and the hardened cement paste. Pore characteristics that permit no penetration of the surface of the aggregate particle are not conducive to a good bond.

Much work relating porosity and durability of cherts has been done by Cantril and Campbell (*6*), Wuerpel and Rexford (*7*), Sweet and Woods (*8*), and Schuster and McLaughlin (*9*). As investigated by Schuster and McLaughlin (*9*), cherts with high porosity are more susceptible to freeze-thaw deterioration than those with low porosity; this further demonstrates that this concept holds for air-entrained concrete as well as concrete with no entrained air. This relationship definitely supports the ideas of Wuerpel and Rexford (*7*), and Sweet and Woods (*8*), but this principle may not hold for shales containing relatively high porosity. Since shale is considerably weaker than the surrounding mortar, it will fail internally because of the pressures developed in freezing without disrupting the mortar.

Other characteristics such as pore size and continuity of aggregate pores are recognized to be more important than total porosity. Lewis and Dolch (*10*) maintained that the harmful pore size is that which is large enough to permit water readily to enter much of the pore space but not large enough to permit easy drainage. Pore size determines the properties of permeability, absorption, and capillary potential. Dolch (*11*) explained: "A rock with larger pores will have a higher absorptivity than one with smaller pores. It will also have a higher permeability and a lower capillary potential. This means that a rock with smaller pores will acquire water more slowly but will retain it longer and more tenaciously than will a rock with larger pores."

Study by Sweet (*12*) has shown that critical pore size for freezing-and-thawing durability of limestone aggregates is about 5 microns. Blanks (*4*) has found that, under natural conditions of freezing and thawing, voids less than 5 microns in diameter, and particularly those less than 4 microns in diameter, will drain effectively only at hydrostatic pressures that exceed the tensile strengths of some rocks and concrete. Sweet (*12*) also noted that in Indiana limestone aggregates the volume of microvoids, (the term "microvoids" refers to voids less than 5 microns in diameter) expressed as a ratio of the total volume, was less than 0.057 for aggregates with good field performance records and greater than 0.091 for aggregates with poor service records. Later work by Schuster and McLaughlin (*9*) showed that the significance of 5-micron pores was questioned: "The freeze-thaw durability of concrete containing chert apparently is not as dependent on pores in the chert less than 5 microns in diameter as has been postulated by Sweet (*12*)." Due to these conflicting conclusions, it seemed desirable to make more thorough research of the relationships of pore characteristics as determined from total porosity and pore size distribution measurements. In this study, attempts were also made to correlate these pore characteristics of aggregates with durability of the concrete when exposed to freezing and thawing.

Testing Program

Results of 45 rock particles from five of the different aggregate sources are reported upon. The aggregates were selected as being representative of the high- and low-durability aggregates covered by Chapters Two and Three. Although the results do not directly affect the conclusions, they do aid in the understanding of the freezing-and-thawing phenomena. Tests run on aggregates reported in the interim report are not shown here as they do not add to the conclusions drawn.

In the study reported here, an Aminco-Winslow mercury-intrusion porosimeter, produced by American Instrument Company, Silver Spring, Maryland, was used and operated up to pressures of 15,000 psi. The pore characteristics determined were total effective porosity, porosities in certain pore size range, pore volumes expressed as percent of total pore volume for certain pore size ranges, and pore modulus. Other pore characteristics of aggregate as a whole were determined by means of specific gravity and absorption tests.

The five aggregates tested were D, E, F, I, and K. From each type of aggregate, several particles were hand-picked to represent the source in the investigation. These hand-picked particles appeared adequately to represent the source mineralogically and in texture. Most of the rock particles used were broken from a large size of rock which was easier to recognize and classify.

Pore-Characteristic Calculations

Total effective porosity, n, is calculated as:

$$n = \frac{V_v}{V_B} \times 100\% \qquad \text{(B-1)}$$

in which V_v is the total mercury-intrusion pore volume or the cumulative pore volume up to 15,000 psi, in cc; V_B, the bulk dry volume of the sample, in cc. Hence, V_B is the ratio of the sample's weight in air to its bulk dry specific gravity. Therefore, total effective porosity can also be calculated by multiplying the total mercury-intrusion pore volume and bulk dry specific gravity of the sample, then dividing by the weight of the sample in air.

Average total effective porosity is the sum of total effective porosity of one aggregate type multiplied by percent of constituent of that type, expressed as a percentage.

The porosity between certain diameters is the product of the cumulative pore volumes in that certain range of diameters and the bulk dry specific gravity of the sample, divided by the weight of sample, expressed as a percentage.

Average porosity between certain diameters is the sum of the porosity between certain diameters of one aggregate type multiplied by percent of constituent of that type, expressed as a percentage.

Percent of total voids volume between certain diameters is the ratio of the cumulative pore volumes in that certain range of diameters divided by the total mercury-intrusion pore volume, expressed as a percentage.

Average percent of total voids volume between certain diameters is the sum of percent of total voids volume between certain diameters of one aggregate type multiplied by percent of constituent of that type, expressed as a percentage.

Pore modulus is the cumulative sum of the penetration per gram corresponding to predetermined diameters expressed in cc per gram. In this study, diameters used were 5, 2, 1.4, 0.6, 0.25, 0.11, 0.05, 0.02, and 0.01 microns.

The porosity between certain diameters and percent of total voids volume between certain diameters are selected. Their pore diameter ranges in microns are: 76-5, 76-0.58, 76-0.046, 76-0.029, 76-0.02, 5-0.012, 0.58-0.012, 0.046-0.012, 0.029-0.012, and 0.02-0.012.

Relationships Between DF_{100} and Aggregate Properties

One hundred cycle durability factors were calculated from 16 beams made from each aggregate source. The results are given in Table B-7. Results of specific gravity and absorption of the three fractions used, ¾" — 1", ½" — ¾", and ¼" — ½", are given in Table B-8. Percent of each type of rock particles in each source is shown in Table B-9. Table B-10 summarizes Tables B-7 and B-8 and gives a combination of the average values. It was found that aggregate having an average specific gravity (saturated surface dry basis) lower than 2.55 produced concrete having significantly lower durability than concrete having aggregate with an average specific gravity higher than 2.60. The specific gravities of aggregate E and aggregate F are 2.55 and 2.42, and their average durability factors are 12.99 and 3.18, respectively. Thus, they fall into the bad and ex-

TABLE B-7

DURABILITY FACTOR OF THE FREEZING-AND-THAWING TESTS

BEAM NO.	DF_{100} AGGR. D	AGGR. E [a]	AGGR. F [a]	AGGR. I	AGGR. K
1	99	3 (6)	3 (6)	97	85
2	99	3 (5)	4 (7)	100	85
3	100	3 (6)	3 (7)	100	85
4	97	3 (6)	4 (8)	100	89
5	97	4 (8)	4 (7)	100	81
6	96	5 (9)	3 (5)	96	85
7	100	5 (11)	4 (8)	100	88
8	97	9 (18)	4 (8)	100	89
9	89	3 (6)	3 (5)	96	89
10	93	3 (6)	3 (5)	95	83
11	93	4 (7)	3 (5)	98	93
12	100	7 (13)	2 (4)	94	88
13	99	44 (88)	3 (7)	96	84
14	96	37 (73)	3 (5)	99	86
15	100	43 (85)	3 (6)	98	86
16	100	35 (69)	3 (6)	98	87
Avg.	97	13	3	97	87

[a] Beam failed at the number of cycles in parentheses.

TABLE B-8

SPECIFIC GRAVITY AND ABSORPTION OF PARTICLES

SOURCE OF SAMPLE	FRACTION OF SAMPLE	BULK SAT. SURFACE DRY SP. GR.	BULK DRY SP. GR.	ABSORPTION (%)
D	¾"-1"	2.62	2.57	2.03
	½"-¾"	2.61	2.54	2.44
	¼"-½"	2.60	2.54	2.54
	Average	*2.61*	*2.55*	*2.34*
E	¾"-1"	2.52	2.42	4.15
	½"-¾"	2.54	2.45	3.68
	¼"½"	2.58	2.50	3.34
	Average	*2.55*	*2.46*	*3.72*
F	¾"-1"	2.43	2.28	6.38
	½"-¾"	2.43	2.29	6.17
	¼"-½"	2.42	2.27	6.41
	Average	*2.42*	*2.28*	*6.32*
I	¾"-1"	2.68	2.64	1.46
	½"-¾"	2.68	2.64	1.60
	¼"-½"	2.67	2.63	1.67
	Average	*2.68*	*2.64*	*1.58*
K	¾"-1"	2.66	2.60	1.78
	½"-¾"	2.65	2.60	1.83
	¼"-½"	2.64	2.59	1.96
	Average	*2.65*	*2.60*	*1.86*

tremely bad category according to the durability factor range described by Walker (*13*). In the bulk dry specific gravity column in Table B-10 the limit for bad aggregate would be approximately 2.45 or lower, since Aggregate E and Aggregate F both fall below the limit, 2.46, and 2.28, respectively. This is in accord with the work of Sweet and Woods (*8*) which indicated low specific gravity chert to be the most susceptible to freeze-thaw deterioration. Both of these aggregates contain significant quantities of chert (see Table B-9).

Comparison of the absorption tests of the five sources to the durability factors of concrete beams made from each source also indicate that a very clear relationship exists. Coarse Aggregates E and F have absorptions of 3.73 and 6.32, respectively, whereas the others are from about 1.6 to 2.4. The absorption of each source is directly related to its effective porosity and its constituents. Table B-11 shows this relationship. The most absorptive rock particles are cherts which have 6.32 percent of absorption and 14.39 porosity. It demonstrates that Aggregate F is not only very absorptive but also rather porous; chert of this type is very susceptible to frost damage, because of its ability to become and stay highly saturated. The durability factor 3.18 of Aggregate F evidently shows this fact. By looking generally at Table B-7, which shows that all beams failed in about 5 to 8 cycles, it is obvious that this chert aggregate failed in a very short period of time under frost action. The second most absorptive aggregate is Aggregate E, which has an absorption of 3.73 percent and 11.41 percent average effective porosity. This high absorption value may be due to 48 percent sandstone with 16.58 porosity and 23.5 percent chert with 8.40 porosity. Although Aggregate D has almost the same amount of sandstone (53.3%) it is much less porous (4.96%), and its chert has approximately the same porosity (8.33%); but is much less in quantity (4.8%); this may cause the absorption to be a low value of 2.34.

As has been shown, it appears that coarse aggregates with vacuum-saturated absorptions of less than about 3.5 percent will not cause too much damage. This agrees with Schuster and McLaughlin's statement about chert (*9*): "Apparently chert with absorptions of about 4 percent or greater will cause freeze-thaw failure when used in amounts as low as 6 percent of the coarse aggregate." As given in Table B-11, the other three sources have low absorption values of 2.34, 1.86, and 1.58. Their corresponding durability factors, 97, 86, and 97, show that these aggregates are very durable and frost resistant.

Freeze-thaw durability of concrete beams containing chert show a significant deterioration when containing high percentage chert from low specific gravity groups (*9*). The

TABLE B-9

PERCENT OF EACH TYPE OF ROCK PARTICLES FROM EACH SOURCE

AGGR.	DESCRIPTION OF ROCK PART.	% OF WHOLE AGGR. ¾ IN.- 1 IN.	½ IN.- ¾ IN.	¼ IN.- ½ IN.	AVG. % OF 3 FRACTIONS OF SEL. AGGR.	AVG. % OF 3 FRACTIONS BASED ON 100 %
D	Siltstone	17.9	17.6	26.9	20.8	21.6
	Shale	5.8	2.3	2.3	3.5	3.6
	Chert	4.0	5.1	4.7	4.6	4.8
	Quarzite	18.6	18.5	11.5	16.2	16.8
	Sandstone	51.2	54.5	48.5	51.4	53.3
	Total	97.5	98.0	93.9	96.5	100.0
E	Siltstone	17.5	18.0	13.5	16.3	20.8
	Chert	13.2	18.4	23.5	18.4	23.5
	Quartz	2.3	1.6	2.3	2.1	2.7
	Quarzite	3.2	4.4	4.0	3.9	5.0
	Sandstone	39.4	37.2	36.3	37.6	48.0
	Total	75.6	79.6	79.6	78.3	100.0
F	Chert				100.0	100.0
I	Dolomite	27.6	28.8	19.5	25.3	30.5
	Limestone	36.9	33.8	39.5	36.7	44.2
	Sandstone	21.7	20.6	20.7	21.0	25.3
	Total	86.2	83.2	79.7	83.0	100.0
K	Limestone				100.0	100.0

results of chert studies given by Schuster and McLaughlin (*9*) show that chert having porosity of nearly 13 percent in the 2.45 minus group cause the most severe freeze-thaw deterioration. Therefore, Aggregate F supports this evidence. From this discussion, it is evident that the durabilities of these concretes decrease as the specific gravities of the aggregates decrease, and as percent of absorption and total effective porosity increase.

Pore Size Distributions

Many early studies have maintained that the durability of aggregates is dependent more on the size and continuity of aggregate pores than on total porosity. In view of this, several of the more significant relationships between pore size distributions are presented and discussed here.

Typical pore-size distribution curves were developed to aid the description of pore characteristics. These curves are shown in Figure B-1. Generally speaking, a large pore modulus and a large porosity will have a high position on the graph, as shown by the top curve. The lower curve represents a low pore modulus, and a low porosity. Also, the top curve with high porosity should require more time to attain any particular degree of saturation than the lower curve with low porosity. The average pore-size porosities of the five sources are summarized in Tables B-12 and B-13.

In looking at Tables B-12 and B-13, for each aggregate source, porosities increase sharply from 76-5 micron through the 76-0.46 micron interval before leveling off. Also, very sharp decreases in porosity data occur, going from the 0.58-0.012 micron through the 0.029-0.012 interval before leveling off. The fine porosity range of 0.58-0.046 microns represents an especially large amount of the available pore space. The corresponding percentages of total voids volume of 0.58-0.046 micron of the five sources have a range from about 21 percent to 63 percent.

By general comparison of the five sources, the porosities of 76-5 microns range of Aggregates D, I, and K are all less than 1 percent; Aggregates E and F have a value of about 2.5 percent. This larger value indicates that Aggregates E and F are more porous in the large pore sizes than the other aggregate sources.

The limit of 5 microns chosen in this study was indicated by Sweet (*12*) as a critical pore size for freezing-and-

TABLE B-10

AVERAGE VALUE OF SPECIFIC GRAVITY, ABSORPTION, AND DURABILITY FACTOR OF EACH SOURCE

SOURCE OF SAMPLE	SG_{BD} [a]	SG_{BSSD} [b]	ABSORP. (%)	DF_{100}
D	2.55	2.61	2.34	97
E	2.46	2.55	3.73	13
F	2.28	2.42	6.32	3
I	2.64	2.68	1.59	97
K	2.60	2.65	1.86	86

[a] Specific gravity bulk dry.
[b] Specific gravity bulk saturated surface dry.

TABLE B-11

AVERAGE VALUE OF PERCENTAGE OF COARSE AGGREGATE, POROSITY, DURABILITY FACTOR

AGGR.	DESCRIPTION OF ROCK PARTICLE	AVG. % OF COARSE AGGR.	AVG. EFFECTIVE POROSITY (%)	AVG. DF_{100}	% OF ABSORPT.
D	Siltstone	21.6	5.84		
	Shale	3.6	3.01		
	Chert	4.8	8.83		
	Quarzite	16.8	4.54		
	Sandstone	53.3	4.96		
	All	100.0	5.19	97	2.34
E	Siltstone	20.8	5.94		
	Chert	23.5	8.40		
	Quartz	2.7	2.26		
	Quarzite	5.0	3.55		
	Sandstone	48.0	16.58		
	All	100.0	11.41	13	3.73
F	Chert	100.0	14.39	3	6.32
I	Dolomite	30.5	5.60		
	Limestone	44.2	2.84		
	Sandstone	25.3	5.22		
	All	100.0	4.29	97	1.58
K	Limestone	100.0	6.73	86	1.86

thawing durability of limestone aggregates. Again, Sweet had noted that in Indiana limestone aggregates the volume of microvoids, expressed as a ratio of the total volume, was less than 0.057 for aggregates with good field performance records and greater than 0.091 for aggregates with poor service records. As given in Table B-12, results of average values of the 5-0.012 micron porosities show that Aggregates D and I, having values of 4.45 percent and 3.34 percent, respectively, are all less than 5.7 percent; Aggregate K, having a value of 6.24 percent, is only a little larger than 5.7 percent but much less than 9.1 percent; Aggregates E and F are both larger than 9.1 percent. According to Sweet's statement, Aggregates D, I, and K are considered to have good and fair field performance records, and Aggregates E and F have poor service records. As compared to durability factor in Table B-7, evidence shows that they do agree with each other.

Pore modulus indicates the relative quantity of large and small pores of a material for a given porosity; that is, a large value would mean numerous pores in the large diameter level, and a lower value would accordingly indicate fewer large pores. As given in Table B-14, results of average pore modulus correlate quite well with porosity. The high-porosity groups of Aggregates E and F have a value of

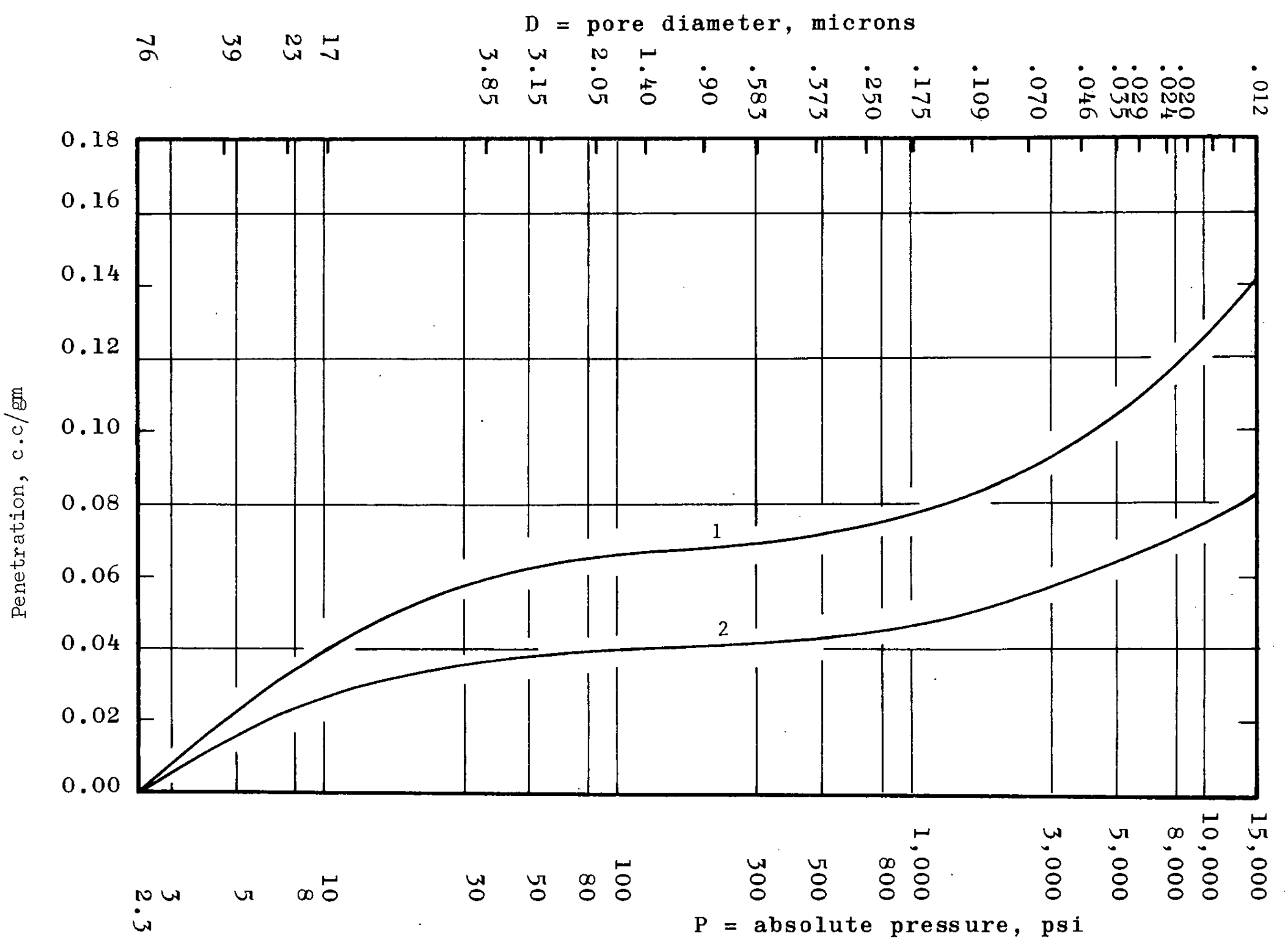

Figure B-1. Typical pore-size distribution curves.

TABLE B-12

AVERAGE POROSITY BETWEEN CERTAIN DIAMETERS

		AVG. POROSITY (%) BETWEEN DIAMETERS (MICRONS)—											
SOURCE	TEP [a] (%)	76-5	76-.58	76-.046	76-.029	76-.02	5-.012	.58-.012	.046-.012	.029-.012	.02-.012	.58-.046	5-.58
D	5.19	0.74	1.50	3.68	4.17	4.46	4.45	3.69	1.51	1.02	0.73	2.18	0.76
E	11.96	2.58	7.56	10.06	10.67	11.24	9.38	4.40	1.90	1.29	0.72	2.50	4.98
F	14.39	2.33	3.98	8.98	11.42	12.74	12.06	10.41	5.41	2.97	1.65	5.00	1.65
I	4.29	0.95	1.30	2.63	3.01	3.44	3.34	2.99	1.66	1.28	0.85	1.33	0.35
K	6.73	0.49	1.05	5.25	5.81	6.21	6.24	5.68	1.48	0.92	0.52	4.20	0.56

[a] Total effective porosity.

TABLE B-13

AVERAGE PERCENT OF VOIDS VOLUME BETWEEN CERTAIN DIAMETERS

		AVG. VOIDS VOLUME (%) BETWEEN DIAMETERS (MICRONS)—											
SOURCE	TEP [a] (%)	76-5	76-.58	76-.046	76-.029	76-.02	5-.012	.58-.012	.046-.012	.029-.012	.02-.012	.58-.046	5-.58
D	5.19	14.2	28.9	70.9	80.4	86.0	85.7	71.1	29.1	19.6	14.0	42.0	14.6
E	11.96	21.6	63.3	84.1	89.2	93.9	78.4	36.8	15.9	10.8	6.0	20.9	41.7
F	14.39	16.2	27.7	62.4	79.4	88.6	83.8	72.4	37.6	20.6	11.4	34.8	11.5
I	4.29	22.1	30.3	61.3	70.1	80.1	78.0	70.0	38.8	30.0	20.0	31.0	8.2
K	6.73	7.3	15.6	78.0	86.4	92.3	92.7	84.4	22.0	13.6	7.7	62.4	8.3

[a] Total effective porosity.

about 0.29 and 0.26, respectively, and the low-porosity groups of Aggregates I, D, and K have a value of about 0.07, 0.09, and 0.10. Comparison of the average pore modulus to durability factors of the five sources indicate that aggregate having an average pore modulus of about 0.25 or higher would have significantly low durability than aggregate having an average pore modulus of about 0.10 or lower. As shown in Table B-14, pore modulus of Aggregates E and F are about 0.29 and 0.26 and their corresponding durability factors are 13 and 3; while pore modulus of Aggregates I, D, and K are 0.07, 0.09 and 0.10 and their durability factors are 97, 97, and 86, respectively.

As compared to absorption, Aggregate F has an absorption of 6.3 percent, and Aggregate E has an absorption of 3.7 percent. Absorption of E is about one-half of F, but porosities are similar (F = 14.39, E = 11.41). However, even though Aggregate F has a higher porosity, it has a lower pore modulus, meaning it has more pores in the fine range than Aggregate E. This is confirmed by examination of Tables B-12 and B-13. The higher absorption of Aggregate F is probably accounted for by its higher porosity, but the greater number of fine pores contributed to a lower durability. Aggregate F is also more likely to retain the water it absorbs because of the fine pores. This significant relationship is supported by Rhoades and Mielenz (*2*) who have shown that the aggregate with the smaller pores would attain and retain a higher degree of saturation and hence be most likely to fail when frozen.

Conclusions

A brief summary of major findings is as follows:

1. The durability of the concretes studied decreased as the specific gravity of aggregates decreased, and as percent of absorption and total effective porosity increased.
2. The correlations between durability and porosity in the fine pores range were quite good. In general, durability decreased as porosity of fine pores increased. Results of the fine porosity range of 5-0.012 micron diameters are in good agreement with Sweet's postulation that critical pore size for freezing-and-thawing durability is about 5 microns.
3. A good relationship exists between durability and pore modulus. The indication for the concretes studied is that aggregates having a pore modulus of about 0.1 or lower have very high durability.

TABLE B-14

PORE MODULUS AND DURABILITY FACTOR

SOURCE OF AGGR.	TOTAL EFFECT. POROSITY (%)	PORE MODULUS (CC/GM)	DF_{100} (%)
D	5.19	0.0947	97
E	11.41	0.2945	13
F	14.39	0.2584	3
I	4.29	0.0693	97
K	6.73	0.1006	86

APPENDIX C

COOPERATIVE TEST PROGRAM

The objectives of both NCHRP Projects 4-3(1) and 4-3(2) were addressed to the same general problem, that of identifying deleterious particles and predicting performance of aggregates when used in portland cement concrete. Free interchange of information was maintained throughout the studies by the involved research personnel at The Penn State University and Virginia Polytechnic Institute. In addition to the over-all cooperative attitude, tests were conducted on several specific aggregate fractions by both agencies and the results are reported herein. (The cooperative test program is also included as Appendix H in *NCHRP Report 66.*)

Six aggregate fractions were selected for the cooperative effort, and all test samples were prepared by Penn State with representative portions being supplied to V.P.I. Tests conducted on the cooperative aggregate samples at V.P.I. were ASTM C290 and the V.P.I. slow freeze. The results are given in Table C-1. The Penn State adaptation of the slow-cooling method and a number of other tests on individual particles were conducted on the selected fractions by Penn State. The resulting data are given in Table C-2.

On the basis of the durability factors being above 80 and $b_t > +1.0$, the V.P.I. tests would indicate that all six aggregates are satisfactory for use in concrete subjected to freeze-thaw. Using the Penn State criteria selected from the slow-cooling method, with a dilation of less than 5 μ in. per cycle indicating a very sound aggregate and a dilation greater than 100 μ in. per cycle a very frost-susceptible aggregate, two of the cooperative aggregates are very sound and the remaining four are in the intermediate range of frost susceptibility.

As indicated in Table C-3, agreement between the Penn State and V.P.I. methods does not appear to be good, possibly because of the narrow frost-susceptibility range of the cooperative aggregate fractions. However, it should also be recognized that the V.P.I. method was not developed for use with aggregate fractions, whereas all of the Penn State research did involve fractions. Because the cooperative aggregates were fractions, no observed field performance information is available for comparison with either the V.P.I. or Penn State methods. Table C-3 does show that the relative rating of the cooperative aggregates by the Penn State rapid test methods agrees quite well with Penn State slow-cooling method ratings.

TABLE C-1

SUMMARY OF V.P.I. TESTS ON COOPERATIVE AGGREGATE FRACTIONS

AGGREGATE	DURABILITY FACTOR ASTM C290, DF_{100}	V.P.I. SLOW FREEZE DF_{100}	b_t
02SLs1	95	97	+2.4
14SSt1+2	98	99	+4.8
16SSs1	98	99	+3.0
18IS+IT	99	99	+3.0
38MGn2	97	100	+7.5
34MAm1	100	100	+3.9

TABLE C-2

SUMMARY OF PENN STATE TESTS ON COOPERATIVE AGGREGATE FRACTIONS

AGGREGATE	VACUUM SATURATED ABSORPTION (%)	SINGLE PARTICLE DILATION (μ IN./IN.)	VOLUMETRIC PARTICLE EXPANSION (%)	AVERAGE PERMEABILITY (DARCYS)	LOG_{10} (PERM. $\times 10^8$) $\times$ V.S. ABS.	ΔL	VOL. % DIL.	DILATION PER CYCLE, SLOW-COOLING METHOD (μ IN./IN.)
02SLs1	2.502	225.	0.33	0.353×10^{-4}	6.319	568.	0.783	48.66
14SSt1+2	1.586	137.5	—	3.764×10^{-5}	2.499	217.	—	11.64
16SSs1	1.391	169.5	0.30	2.053×10^{-5}	1.826	213.	0.394	10.00
18IS+IT	1.995	176.8	—	7.732×10^{-6}	1.737	157.	—	6.09
38MGn2	1.712	246.8	0.17	2.941×10^{-6}	0.802	116.	0.079	3.08
34MAm1	0.221	23.8	0.08	5.663×10^{-7}	−0.056	−6.	−0.018	0.87

TABLE C-3

RATING OF COOPERATIVE TEST FRACTIONS BY PENN STATE AND V.P.I. METHODS

TEST METHOD	FROST-SUSCEPTIBILITY RATING [a] 02SLs1	14SSt1+2	16SSs1	18IS+IT	38MGn2	34MAm1
Penn State:						
Slow-cooling method, concrete specimens	1	2	3	4	5	6
Rapid tests:						
$\log_{10}$ (perm. $\times 10^6$) $\times \Delta$ length change	1	2	3	4	5	6
$\times$ V.S. absorption	1	2	3	4	5	6
V.S. absorption	1	4	5	2	3	6
Vol. percent dilation	1	—	3	—	5	6
$\times \log_{10}$ (perm. $\times 10^6$)	1	—	3	—	5	6
V.P.I.:						
ASTM C290	1	3–4	3–4	5	2	6
Slow-freeze method	1	5	2–3	2–3	6	4

[a] Decreases from 1 to 6.

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