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Tests for Concrete and Durability of Concrete Aggregates

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Contents

A Field Investigation of the AE-55 Air Indicator

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The results of a statewide experiment to compare the AE-55 air indicator for concrete with conventional pressure methods are presented. The data from 835 comparative tests with various materials and operators are statistically analyzed and compared with results of limited laboratory studies previously given by Grieb and Mather (HRB Bull. 176). The results of the present study are in agreement with previous work and give a field verification of laboratory data.

From the comparison some estimate of the reliability can be made, as well as the effect of using an unscreened sample. As a result of this study the indicator is being supplied to inspectors for use in control of air in concrete.

●THE rapid determination of the properties of concrete and concrete materials has received increased attention in recent years as evidenced by the development of such items and procedures as the Kelly ball penetrometer for determining consistency, the impact hammer for estimating strength, and the quick chemical test for alkali reactivity of aggregates. A method to replace the sometimes laborious procedure for determining air content of concrete has recently received attention as a result of the patenting of a relatively simple device, designated the Chace AE-55 Air Indicator, which uses the principle of volumetric displacement of entrained air from a small mortar sample.

The method was developed to afford a means of air content determination which, although somewhat less precise than more conventional methods, would enable an inspector to perform more tests and thus exercise closer control of concrete uniformity. Because of its simple design, the AE-55 is less susceptible to mechanical difficulties than is more conventional equipment although it is susceptible to breakage. The method is not intended to replace pressure, volumetric, or gravimetric methods for laboratory determinations, but rather to serve as an aid in field control. Its small size, low cost, and convenience have caused favorable comment concerning its use.

The indicator which is composed of two sections, is shown in Figure 1. One part is a glass cylinder similar to a filtration crucible holder. This cylinder is about 1 in. in diameter and 3 in. long and tapers to a stem ¼ in. in diameter and 3 in. long. This stem is marked by 11 equally spaced graduations. The second part of the indicator is a brass cup approximately ¾ in. in diameter by ½ in. deep. This cup is attached to a rubber stopper which fits into the glass cylinder. Either of two types may be supplied by the manufacturer. As shown in Figure 2, the more recent type, designated "B" and shown in the lower part of the figure, has a ground fluted end, whereas the older type "A" has a plain end.

A determination of air content may be made in approximately 2 to 3 min. using about 3.7 ml of mortar from the concrete mix. The test procedure consists of securing a sample of mortar from the mix and rejecting material retained on a No. 10 mesh sieve. Many times in actual practice this screening is omitted and the sampling consists of working excess mortar to the surface and removing it with the fingers or a small trowel,

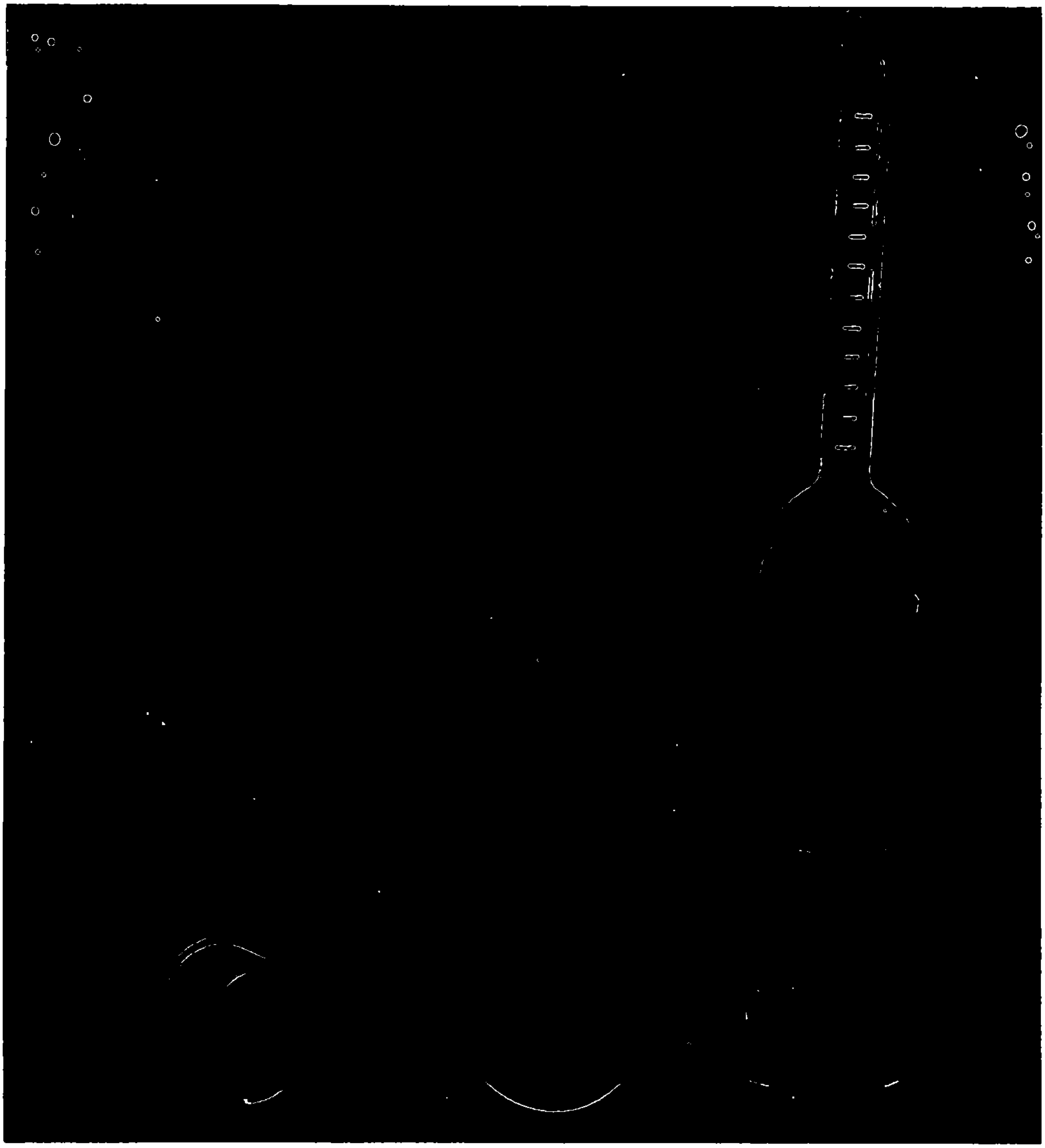

Figure 1. Chace AE-55 air indicator.

rejecting any obviously large particles. The brass cup is filled with this material and rodded with a small wire (normally a paper clip). The mortar is struck off and the cup placed in the glass tube which has previously been filled to the designated level with alcohol. The device is then gently agitated to remove entrained air and the difference in alcohol level is recorded in terms of number of spaces. Based on the mortar content of the concrete mix, a correction factor is applied to the reading to obtain the air content of the mix. The complete test procedure and correction factors supplied by the manufacturer are contained in the Appendix.

For several years the indicator has been given limited use by inspectors and materials engineers on various projects throughout Virginia as well as in other states. While little quantitative data are available, the reaction to the indicator generally has been favorable.

Published data concerning comparisons of the AE-55 indicator with other methods are not extensive. In 1956, Grieb reported a study of the accuracy of the AE-55 conducted in the laboratory of the Bureau of Public Roads (1). The test series consisted of determining the air content of 84 different laboratory concrete mixes by means of

Figure 2. Available types of AE-55 indicator, type "B" on left.

pressure and gravimetric methods and comparing the results with those obtained with the AE-55 indicator. These mixes utilized different cements and aggregates, and air contents ranging from 1 to 9 percent as recorded by the pressure method. The values obtained with the AE-55 indicator by two different operators were in good agreement with those obtained by means of the pressure method for air contents between 3 and 7 percent. For values of air content less than 3.0 percent as determined by the pressure meter, the AE-55 gave results averaging about 1.0 percentage point high. For air contents of more than 6.0 percent the AE-55 indicator gave values averaging somewhat more than 1.0 percentage point low. Grieb presented a correction curve indicating the amount by which the AE-55 should be corrected to bring the readings into agreement with the pressure values. It should be noted that the samples in these tests were taken without screening and that the mortar correction was not applied to the data.

Tests of 107 batches of concrete were made at the Waterways Experiment Station and reported by Willetts and Kennedy (2). The results of these tests were in substantial agreement with those reported by Grieb with the exception that the differences between the AE-55 and pressure values at high air content were not as large. The samples in this study were screened through a No. 10 sieve.

Early work with the AE-55 used in the concrete laboratory of the Virginia Council of Highway Investigation and Research substantiated the results of both the above studies for air contents between 4 and 7 percent. In these tests different operators made only one determination on an unscreened sample, and it was found that 85 percent of the time the air content as determined with the AE-55 was within ± 0.5 percentage point of the line of average relationship established for the AE-55 and the pressure method (3). Only one determination out of 38 varied by more than one percentage point.

Following the laboratory work the AE-55 was utilized in connection with a study of paving mixers conducted during the summer of 1958 on two different projects, one using type III cement and a gravel aggregate, the other a crushed stone and type II cement (4). Again with different operators making only one determination it was found that the air content determined by the AE-55 was within ± 0.5 percentage point of that obtained by the pressure method 84 percent of the time based on the distance of the readings from the line of average relationship. All of the 32 determinations were within 1 percent of the average line.

The general agreement of the results of these preliminary investigations with those of previous studies was encouraging, but it was felt that sufficient data were not available to warrant conclusions regarding the reliability of the device. It appeared from a study of the results that the mortar correction supplied by the manufacturer might not be as important for unscreened samples as a correction related to the percentage of air observed with the AE-55.

To investigate further the reliability of the AE-55 indicator an experiment was designed to provide for a statewide test of the device by the personnel normally charged with the responsibility for determining air content. It was felt that such a coordinated test would result in a quick and definitive evaluation of the device and its possible application for highway use under various conditions. For accomplishing this evaluation a study was conducted cooperatively by the Field Forces of the Virginia Department of Highways and personnel of the Virginia Council of Highway Investigation and Research.

The purposes of this study were:

1. To evaluate the AE-55 air indicator as a field device for the determination of the air content of structural and paving concrete mixes.
2. To compare the AE-55 indicator with conventional pressure methods.
3. To investigate any correction factors which might be necessary to make the AE-55 determinations consistent with pressure meter readings.

TEST PROGRAM

Many practical considerations affected the design of the test because it was desired that it be conducted in connection with normal concreting operations and with a minimum of interruptions thereto. Variations due to operator error, materials, and biases of various kinds could not be practically eliminated but a study of the results indicates that these items did not affect the over-all reliability of the data in any significant degree.

While data were secured from several sources simultaneously, for ease of presentation the study is divided into two parts. Series I is a laboratory investigation of the AE-55 indicator. This study was conducted in connection with other laboratory projects in that the air contents of test batches were determined by means of the AE-55 and pressure methods. Since the mixes tested were not designed to have extreme air contents, the range of air contents was rather small, 3.0 to 7.6 as measured by the pressure method. For each batch tested, two determinations of the air content were made with the pressure meter as required by ASTM C231-56T, while one determination was made with the AE-55. Various operators were utilized during the testing program. The aggregate used in the concrete mixes was not changed and seven brands of type II cement were employed. One hundred and three batches were sampled.

Series II consisted of the studies on regular construction projects conducted cooperatively by Field and Research personnel. Tests were performed in connection with normal construction operations by the inspector assigned to the job, following a uniform procedure, while utilizing his normal schedule and equipment, that is, the meter currently in use was employed in addition to the AE-55. The pressure meters employed were calibrated prior to their use in the testing program.

From the batches designated for test, two samples were secured. On each sample, a pressure meter was used to make two determinations of air content in accordance with ASTM C231-56T, while two determinations were also made with the AE-55 indicators. This procedure was repeated until 40 samples had been tested, giving 80 comparative determinations.

On certain projects it was necessary that more than one operator perform the determinations. While the variation due to operators was not considered a variable in the design of the experiment it is possible from the data to evaluate certain operator differences for specific jobs.

A summary of the pertinent information concerning each project in Series I and II is given in Table 1. Throughout the report reference is made to the projects by numbers. It will be noted that projects one through nine involved a large number of samples and are referred to as major projects. Projects 10 through 13 were minor projects on which it was not possible to run a complete set of tests. Project nine includes all work done in Series I.

ANALYSIS

The data obtained from the comparative trials in Series I and II were analyzed by accepted statistical procedures. A regression analysis was performed on the uncorrected data from each individual project and for all of the projects considered as a whole. A linear regression was used since preliminary tests showed that a curvilinear regression was not necessary.

TABLE 1

DESCRIPTIVE DATA FOR TEST PROJECTS

No	Date of Tests	No. of Tests	Type of Meter (age)	Operators	Mortar Content Cu Ft/Cu Yd	Type CA	Type FA	Range of Air Content (%)
1	4-1-59 4-8-59	80	Protex (1)	3	14	C.S	N S.[1]	2.2-5.8
2	5-19-59 7-28-59	80	Protex (new)	1	15	C.S.	S.S.[2]	4.6-7 7
3	5-20-59 7-7-59	80	Protex (new)	5	15	C.S.	S S	3 0-5 8
4	6-3-59 6-15-59	80	Protex (1)	3	14	Gr	N S	3 6-5 9
5	5-15-59 5-20-59	80	Washington (5)	3	15	Gr	N S.	3.2-4 6
6	5-23-59 7-14-59	80	Protex (1)	4	15	C.S	N.S.	3.2-5.9
7	9-24-59 9-25-59	72	Washington (8)	2	14	Gr	N S	0 5-7.0
8	7-14-59 11-2-59	80	Protex (1)	5	14	C S.	N S	3 0-7.0
9	3-19-58 Present	103	Protex (1)	3	13	Gr	N S.	3.0-7 6
10	5-22-59 8-6-59	48	Protex (?)	4	14	C.S	N S.	3 0-5.8
11	6-19-59 7-23-59	18	Protex (new)	1	14	C.S.	N.S.	3.2-4.7
12	10-1-59 Present	14	Protex (?)	2	15	C.S	S S	2.3-5.6
13	5-19-59 6-25-59	20	Protex (2)	1	15	Gr.	N.S.	3.2-4 7

[1]Natural sand
[2]Stone sand

A correction similar to that of Grieb (1) was prepared from the uncorrected data, and the data were corrected by means of this curve with and without application of the mortar correction. Similar regression analyses were performed on these data. The uncorrected readings were used since indications are that the mortar correction is not applicable to observations on unscreened samples.

In addition to being used in the regression analyses, the data were grouped by ranges of air content as determined by the pressure method and the deviations of the individual AE-55 readings from those obtained with the pressure meter were determined. From the data the various statistical quantities were computed for both corrected and uncorrected cases which enabled a direct comparison with values of previous investigators.

RESULTS

The results of the regression analysis are shown in Figure 3 in which the line of average relationship is given for the uncorrected data from all projects in Series I and II, along with lines denoting the standard error of estimate. The line of average relationship is expressed by the equation $Y = 0.95 + 0.749X$, in which Y denotes the AE-55 reading and X the corresponding reading obtained by the pressure meter. The area bounded by standard error lines (0.5 percent of air) contains 72.9 percent of the 835 readings while 94.0 percent of the readings are within 2 standard errors (1 percent of air). It should be noted that many of the points represent more than one determination. Also, 72 percent of the readings fall within 0.5 percent of air based on the line of equality, rather than the line of average relationship, and within 1.0 percent 92 percent of the time. The number of extreme points falling above the upper limit was approximately equal to the number falling below the lower limit when considered from the line of average relationship; however, considered from the line of equality, the number falling below was considerably greater than that falling above.

It can be seen that the line of regression indicates that at low air contents the AE-55 tended to give higher readings than the pressure meter while at high air contents the AE-55 generally read lower. The reason for this tendency of the AE-55 to read high at low air contents and low at high air contents is not definitely established but it is felt that the tendency is related to the method of sampling. The meter was intended to be utilized with a screened sample which would be relatively uniform. The taking of the sample without screening is influenced by the consistency of the mix, since mixes of low air content are stiffer than comparable mixes having a higher air content. Samples taken from the latter mixes would have a larger proportion of water and would thus tend to give lower air contents. Samples taken from the stiffer mix would tend to have a higher proportion of solid constituents and thus would be expected to give higher air contents. It is also possible that the rodding of the stiffer mixes did not give consolidation comparable to that for the wetter mixes and resulted in more entrapped air.

The lines of average relationship based on the uncorrected data for the nine major projects are shown in Figure 4. The significant fact to be gained from this plot is that the tendency of the meter to read high at low air contents and low at high contents was found for eight of the nine projects. The consistency of the indications from project to project would indicate that a correction curve could be prepared which would correct the readings to equivalent pressure meter values.

Although certain limitations of the method are apparent, the approach of Grieb was followed by which the AE-55 indications were grouped according to pressure readings as given in Table 2.

The difference in average air content as determined by the AE-55 and the pressure method is plotted as a function of the average pressure reading in Figure 5. For comparison, values obtained by previous investigators are shown. Considering all of the factors which could affect the results, the agreement between the data obtained by the

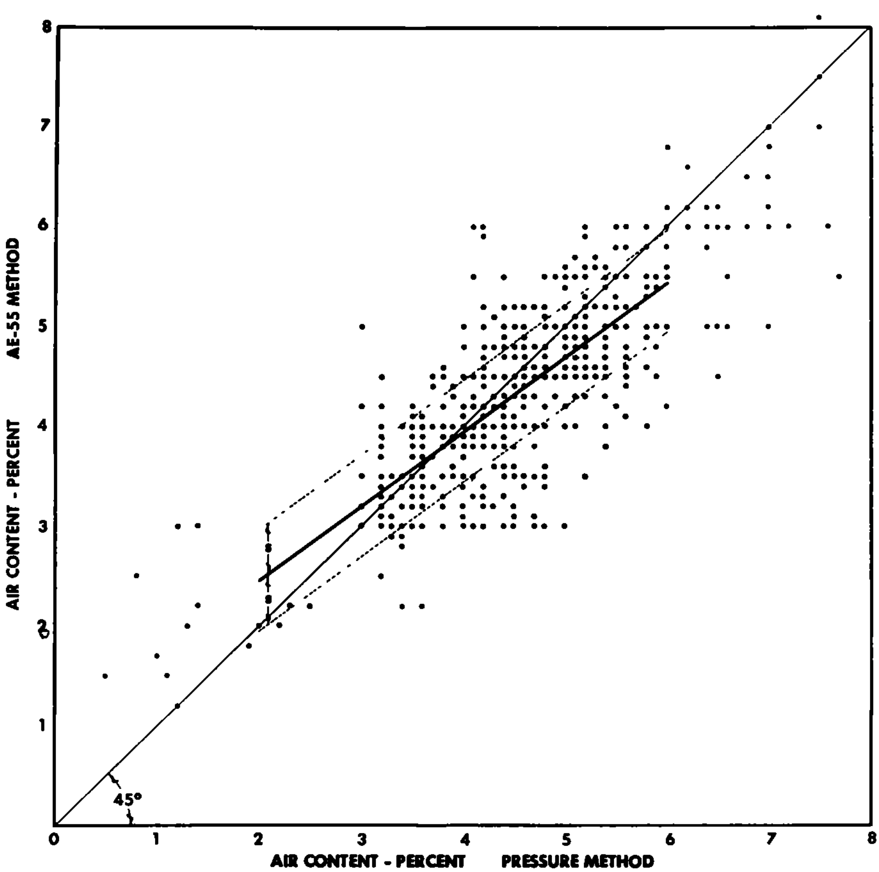

Figure 3. Plot of uncorrected determinations, Series I and II.

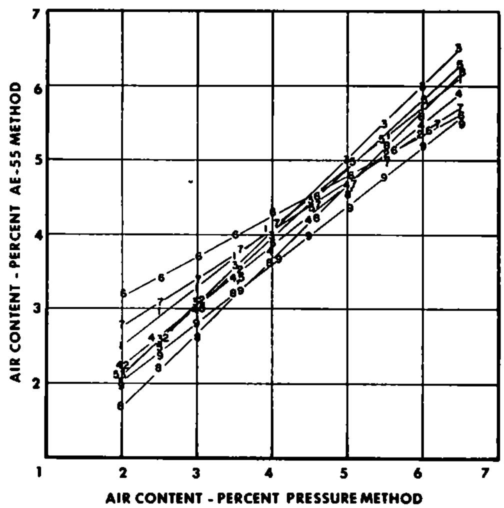

Figure 4. Regression lines from nine major projects, uncorrected data.

TABLE 2

DATA GROUPED BY PRESSURE READINGS

Range (pressure method)	No. of Samples	Average Air Content (%) Pressure Meter	AE-55	Difference	Range of Differences	Std. Dev.
0.0 - 0.9	2	0.6	2.0	+1.4	+1.0 to +1.7	0.495
1.0 - 1.9	9	1.3	2.2	+0.9	-0.1 to +1.8	0.726
2.0 - 2.9	5	2.2	2.1	-0.1	0 to -0.3	0.114
3.0 - 3.9	181	3.5	3.6	+0.1	-0.8 to +2.0	0.305
4.0 - 4.9	328	4.4	4.2	-0.2	-1.8 to +1.9	0.500
5.0 - 5.9	255	5.3	4.9	-0.4	-2.0 to +0.8	0.448
6.0 - 6.9	38	6.2	5.6	-0.6	-2.0 to +0.8	0.638
7.0 - 7.9	17	7.2	6.4	-0.8	-2.2 to +0.6	0.672

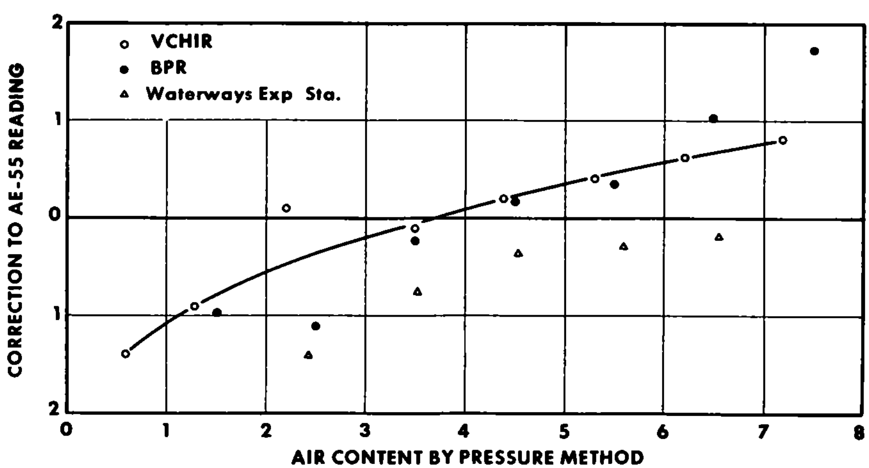

Figure 5. Correction as a function of the pressure method.

different laboratories is striking, especially that between the data from this test and the data from the Bureau of Public Roads test. It should be kept in mind that these two studies were made on unscreened samples. The shape of the curve for the data obtained by Willetts (2) on screened samples is similar but a higher AE-55 air content is indicated as would be expected. The large difference between the values at 2 to 3 percent air is probably a result of an insufficient number of samples. It should be noted that the data from this study represent more samples than do those of previous studies.

The results indicate that the data obtained in independent investigations are compatible and that some correction curve which would allow AE-55 readings to be expressed as equivalent pressure readings might be of value. Figure 6 contains the correction necessary to cause agreement between the AE-55 and pressure readings plotted as a function of the AE-55 indications. Again the readings of previous investigators are shown for comparison. As stated previously, the correction curve for this study is in substantial agreement with that of the BPR study. While the number of data points are limited it will be noted that the WES study, which involved a screened sample, does not show the same trend. It is probable that the WES data would be corrected best by a constant factor.

A curve of best fit was constructed through the points and the corrections indicated by this curve were applied to the data both uncorrected and after application of the mortar correction. The regression lines obtained from the data in Series I and II for the four conditions are shown in Figure 7 along with values for the coefficient of correlation and standard error of estimate. It will be noted that the application of the curve correction did not affect the degree of correlation significantly but reduced the precision as measured by the error of estimate. It would appear that the uncorrected data gave a better measure of air content than attempts at correction for the projects considered as a whole.

The coefficients of correlation, r, and standard errors of estimate, s, for each project are given in Table 3. In the cases where values are not given the mortar content of the mix was 15.0 cu ft/yd and the mortar correction was not applicable. A study of the results from the major projects of Series I and II given in Table 3 will show that the degree of correlation was reasonably constant among projects with the exception of project six on which the coefficient of correlation was 0.58. The data from this project were studied carefully and it was determined that the total number of tests was divided among several operators. When 40 tests conducted by one operator were considered, it was found that his data showed a correlation coefficient of 0.85 and a standard error of 0.24. The results obtained by the other operators showed a lower coefficient.

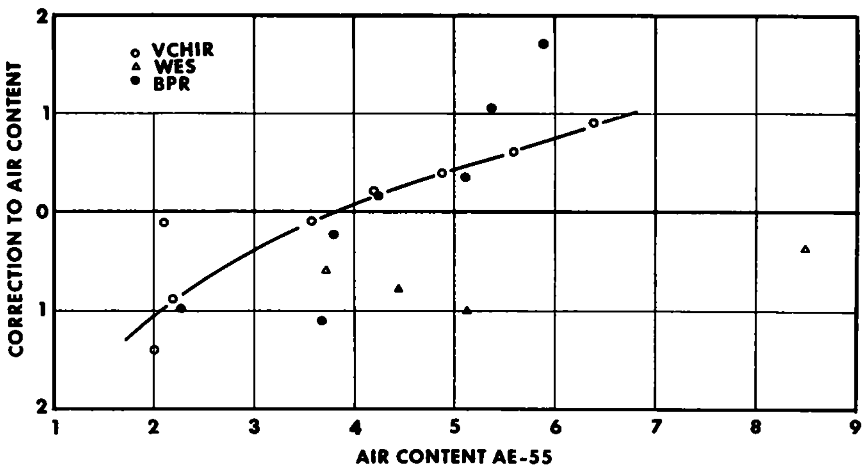

Figure 6. Correction as a function of the AE-55.

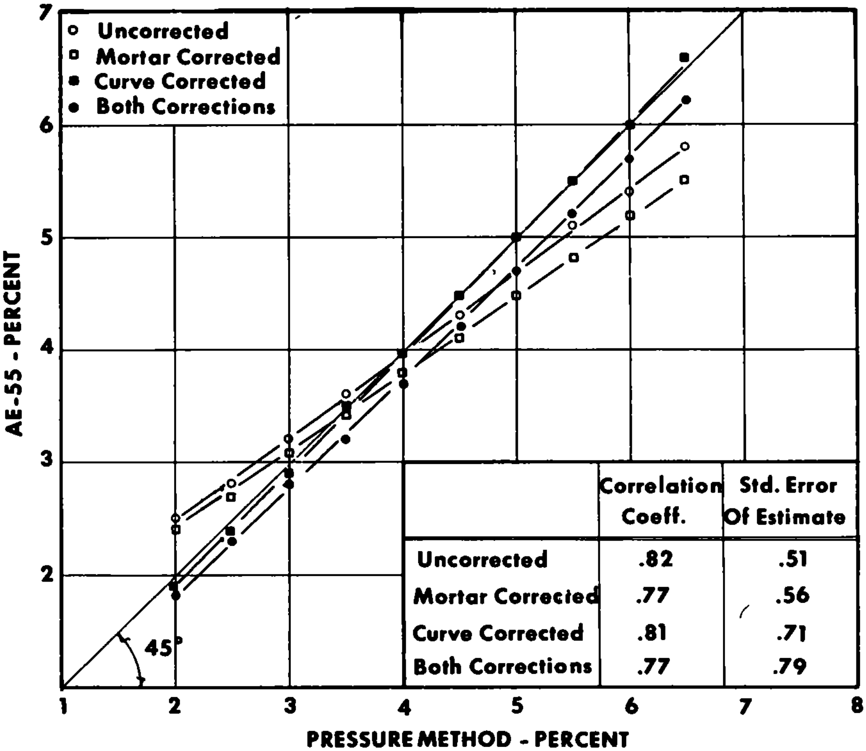

Figure 7. Regression lines from all projects, Series I and II.

TABLE 3

RESULTS OF ANALYSES—SINGLE OBSERVATION

Project	No. of Samples	Uncorrected r	Uncorrected s	Mortar Corrected r	Mortar Corrected s	Curve Corrected r	Curve Corrected s	Both r	Both s
1	80	0.78	0.50	0.78	0.45	0.79	0.71	0.79	0.67
2	80	0.85	0.31	-	-	0.93	0.29	-	-
3	80	0.92	0.29	-	-	0.91	0.41	-	-
4	80	0.69	0.47	0.69	0.44	0.62	0.71	0.69	0.62
5	80	0.89	0.17	-	-	0.72	0.38	-	-
6	80	0.58	0.47	-	-	0.59	0.64	-	-
7	72	0.88	0.67	0.74	0.88	0.88	0.97	0.89	0.90
8	80	0.78	0.59	0.78	0.56	0.78	0.82	0.78	0.77
9	103	0.74	0.61	0.72	0.62	0.73	0.86	0.72	0.89
10	48	0.85	0.39	0.85	0.36	0.85	0.54	0.81	0.60
11	18	0.93	0.18	0.95	0.17	0.93	0.27	0.93	0.26
12	14	0.89	0.55	-	-	0.92	0.66	-	-
13	20	0.59	0.33	-	-	0.58	0.48	-	-
1-13	835	0.82	0.51	0.77	0.56	0.81	0.71	0.77	0.79
1- 9	735	0.80	0.52	0.76	0.57	0.80	0.73	0.76	0.81
1- 8	632	0.83	0.48	0.81	0.50	0.83	0.67	0.80	0.70

Similar analyses of the other projects showed no such variation among operators since apparently the techniques utilized by the various operators were the same.

While no specific operator variable was included in the experiment, the consistency of the data among projects would seem to indicate that the effect of operator variations is not great. It was obvious during the study that a certain amount of technique was necessary in using the AE-55; however, it was observed that once an operator got a feel for the device, through comparative readings with a pressure meter, his accuracy improved. There is no reason to believe that the differences among operators will be great provided an established procedure is followed carefully.

It will also be noted from Table 3 that the mortar correction had little effect on the correlation coefficient as would be expected. The precision as measured by the standard error of estimate was improved slightly in five of the seven cases. However, in only one case did the application of the mortar correction change the standard error by as much as 0.1 percent of air, and this was an adverse change.

It would be expected that samples taken without screening would be less uniform than screened samples and so more variation in the readings would be expected. Because the mortar correction is intended to correct for the fact that larger particles are not included in the sample, it would follow that the tendency for large particles to be included in the unscreened sample would make application of the mortar correction questionable. It appears that the mortar correction supplied by the manufacturer is not applicable for unscreened samples.

The correction made by using the curve shown in Figure 6 also resulted in little change in the accuracy as measured by the correlation coefficient. Furthermore, the precision was increased only slightly in one case and was significantly reduced in many. Based on the average air contents given in Table 5, the correction was beneficial in five of the eight cases. The application of both corrections reflected the effect of each individual one. This does not mean that such corrections are not necessary. To the contrary, the consistency of the trends shown in Figures 4 and 5 would indicate the desirability of such corrections. It is felt that the curve shown in Figure 6 needs modification at the extreme values because of the small number of samples on which it is based. It was noted that application of the curve correction for readings in the middle portion generally had a beneficial effect.

TABLE 4

RESULTS OF ANALYSES OF DATA—AVERAGE OF TWO OBSERVATIONS

Project	No. of Samples	Uncorrected		Mortar Corrected		Curve Corrected		Both	
		r	s	r	s	r	s	r	s
1	40	0.81	0.48	0.83	0.55	0.82	0.66	0.81	0.63
2	40	0.92	0.23	-	-	0.91	0.30	-	-
3	40	0.95	0.24	-	-	0.94	0.33	-	-
4	40	0.76	0.40	0.74	0.39	0.73	0.58	0.75	0.53
5	40	0.93	0.14	-	-	0.94	0.18	-	-
6	40	0.63	0.43	-	-	0.64	0.57	-	-
7	36	0.89	0.65	0.89	0.58	0.89	0.91	0.89	0.90
8	40	0.80	0.57	0.81	0.57	0.80	0.80	0.82	0.58
1-8	316	0.85	0.45	0.83	0.46	0.86	0.61	0.82	0.63

From information supplied by the personnel who performed the tests, it appears that a determination can be made in from two to five minuted with the AE-55, whereas a similar determination with a pressure method would take from 15 to 20 min. It would seem then that it would not be burdensome to require two AE-55 determinations on each sample. In order to investigate the effect of two determinations on accuracy

TABLE 5

AVERAGE AIR CONTENTS DETERMINED FOR FIELD PROJECTS

Project	Samples	Pressure	Uncorrected	Mortar	Curve	Both
1	40	4.52	4.50	4.41	4.71	4.24
2	40	5.24	4.95	-	5.36	-
3	40	4.34	4.39	-	4.59	-
4	40	4.47	4.23	3.93	4.39	3.93
5	40	3.94	3.85	-	3.91	-
6	40	4.77	4.68	-	5.00	-
7	36	4.66	4.52	4.20	4.67	4.26
8	40	5.21	4.86	4.73	5.21	4.72
1-8	316	4.64	4.50	4.40	4.73	4.50

and precision, the average of two AE-55 determinations from the same sample was compared with the average of the corresponding pressure values for the same sample. The results of this analysis are given in Table 4. The data from the laboratory study were eliminated since no repeat determinations were made. Comparison of these correlation coefficients with those in Table 3 will show that in almost every case the effect was to increase both the accuracy and precision although some of the increases were very modest. Thus it appears that a repeat determination would be desirable.

Aside from the consideration of the correlation existing for the various projects, it is interesting to note from Table 5 that the average AE-55 air content on the major field projects differed from that determined by the pressure method by a maximum of 0.3 percent of air. The data in Table 5 are for the average values; however, the same project average would be found if the individual readings were used in computing the average.

The results of the field tests were most encouraging and resulted in the use of the AE-55 indicator by inspectors on jobs throughout the state. Several questions relative to the use of the indicator still warrant study however. Two of the most important are the necessity for using a screened sample and the effect of using the different types of indicators shown in Figure 2.

From the results of the tests reported in this paper as well as previous work (1) it appears that determinations made on samples taken without screening give a sufficiently accurate indication of the air content. It has been found from additional laboratory tests that air contents determined from screened samples are generally higher than those obtained either from unscreened samples or from pressure methods, even when the mortar correction is applied.

All tests reported in this paper were made with the plain (type A) indicator. Additional laboratory tests have indicated that the fluted type of indicator (type B) gives a higher air content than does the plain type. Additional study is needed in this area.

CONCLUSIONS

From the field and laboratory studies as well as consideration of previous studies the following conclusions appear justified.

1. The AE-55 indicator is a reasonably accurate, moderately precise device which is adequate for field measurement of air content of concrete.
2. Under field conditions the AE-55 determination requires about ¼ to ⅓ the time of a pressure determination. In addition to the money represented by this saving, it is felt that the performing of more tests will result in better control of entrained air.
3. Based on a large number of samples, the average air content as determined by the AE-55 will be within ½ percentage point of air as compared with the pressure method.
4. A repeat determination is advisable.

5. While the preparation of the mortar sample by screening would possibly result in more uniform determinations, the results of these tests indicate that for field use sufficient accuracy is obtained with unscreened samples.

6. From this study it appears that the air content as determined on an unscreened sample will be within ½ percentage point of air as determined by pressure methods approximately 70 percent of the time and within 1 percentage point of air about 95 percent of the time based on the correlation line given in this study.

7. If the sample is screened, application of the correction based on the mortar content is possibly desirable.

8. If the determination is made on an unscreened sample, the mortar correction is not necessary.

9. For unscreened samples, the AE-55 indicator tends to give high readings at low air contents and low readings at high contents. Within the range of 3 to 7 percent air there appears to be little difference between the AE-55 and the pressure method.

10. The fragility of the device is a definite disadvantage but this is partially offset by its low cost and freedom from mechanical defects.

ACKNOWLEDGMENTS

The help of all district materials engineers and inspectors who participated in the study is gratefully acknowledged.

The help of Dr. Marvin Tummins and Dr. Bert Goldman in certain phases of the analysis of the data has been invaluable.

The work was conducted under the guidance and with the constant interest of Tilton E. Shelburne, Director of Research.

REFERENCES

1. Grieb, W. E., "The AE-55 Indicator for Air in Concrete." HRB Bull. 176 (1958).
2. Willetts, C. H., and Kennedy, T. B., "A Limited Investigation of the Chace Air Meter." Waterways Experiment Station, Miscellaneous Paper No. 6-189 (Nov. 1956).
3. Newlon, H., Jr., and Morgan, A. H., Jr., "The Effect of Commercial Retarders on Some Properties of Concrete—Progress Report No. 1." Virginia Council of Highway Investigation and Research, Charlottesville (March 1959).
4. Morgan, A. H., Jr., and Newlon, H., Jr., "The Mixing Efficiency of 34-E Dual Drum Pavers." Virginia Council of Highway Investigation and Research, Charlottesville (March 1959).

Appendix

MANUFACTURER'S DIRECTIONS FOR USE OF AE-55 AIR INDICATOR[1]

Fill metal cup with cement mortar paste, excluding particles larger than No. 10. Use a narrow blade to pick up mortar. Do not wet screen. Rod material in cup to compact mortar. Strike off excess even with top of cup.

Hold finger over stem opening and fill large end with isopropyl alcohol to line on glass (alcohol may be inserted in the stem opening after stopper is inserted, with syringe or dropper if desired).

Insert stopper in tube, invert indicator and adjust liquid level to top line of stem making sure that all air bubbles are removed and that the stopper is firmly inserted.

Place finger over stem opening to prevent loss of any liquid and gently roll the indicator from vertical to horizontal several times until all the mortar has been dissolved out of the cup into the alcohol.

With indicator in vertical position carefully remove the finger from the opening

[1]Pat. pending.

and count the number of spaces from the top to the new liquid level. In the case of mixes with 15 cu ft of mortar the number will directly represent the percentage of entrained air in a cubic yard of the concrete. For different mortar content refer to the table below.

When ready to empty the instrument, care should be exercised to invert the glass to flush out particles of sand from between the glass and metal to prevent jamming when removing the stopper.

Wash and clean the assembly immediately after use with clean water.

CONVERSION TABLE

For following mortar contents per cubic yard multiply the stem readings by the following constants:

10 c.f. by 0.67	19 c.f. by 1.26
11 c.f. by 0.73	20 c.f. by 1.33
12 c.f. by 0.80	21 c.f. by 1.39
13 c.f. by 0.86	22 c.f. by 1.46
14 c.f. by 0.93	23 c.f. by 1.52
15 c.f. by 1.00	24 c.f. by 1.59
16 c.f. by 1.07	25 c.f. by 1.66
17 c.f. by 1.13	26 c.f. by 1.72
18 c.f. by 1.20	27 c.f. by 1.78

Investigation of the Impact-Type Concrete Test Hammer

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This work describes a series of tests designed to determine the reliability of results, the feasibility of use, and the practical applications of the test hammer in construction control. Test results are compared to the published findings of other investigators and the reliability of calibration curves under various test conditions is carefully investigated. Indicated strengths are significantly affected by specimen size, restraint or clamping in testing machine, surface texture, mix proportions, and type of aggregate. Coefficient of variation over a wide variety of specimens average 18.8 percent and exceeded 30 percent for some groups of specimens. It is recommended that special calibrations be provided for each mix or change of aggregate, and that use of the test hammer on weak or young concrete be kept to a minimum because such testing may produce significant surface blemishes.

●THE first test series consisted of obtaining hammer rebound values for concrete cylinders selected at random from those being tested during the routine testing program. This series consisted of two hundred 6- by 12-in. and twenty-six 18- by 36-in. concrete cylinders, ranging in age from 28 days to 1 year and older, and varying in weight, curing conditions, water-cement ratios, air contents, cements contents, pozzolans, and aggregates. All cylinders were tested for compressive strength; thirty-two of the 6- by 12-in. and six of the 18- by 36-in. cylinders were also evaluated for modulus of elasticity.

Test hammer readings were obtained with the specimen in an upright position and the hammer held horizontal and normal to the surface of the specimen. The instrument was held firmly as the pressure was gradually increased until impact. Readings were taken within the center two-thirds portion around the cylinder. Care was taken so as to avoid obvious air pockets, honeycomb, and the immediate areas of previous impacts. Specimens were free from restraining load during the hammer testing, but were supported by hand immediately behind the impact area (Fig. 1).

The average rebound value "R" for each specimen was determined from the best suited 10 of 15 readings, (as per manufacturer's instructions, 10 readings nearest average of 15) as recommended in the booklet of operating instructions furnished by the manufacturer of the test hammer.

The second test series consisted of obtaining hammer rebound values on four 6- by 12-in. concrete cylinders under restraining load conditions. An average "R" was determined for each cylinder in an unrestrained condition in the same manner as outlined in the first test. Each cylinder was then placed in the compression machine, and a constant load was maintained while another average "R" determination was made. Average "R" values were determined for each cylinder at five successively increasing constant loads (Figs. 1 and 2).

The third test series was designed to determine the possible use of the test hammer on concrete at early ages, and to measure variations in the rebound value due to different

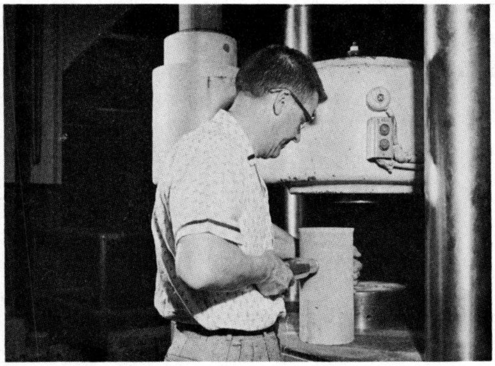

Figure 1. Testing an unrestrained specimen with the concrete test hammer.

aggregate, surface textures, restraining load, and surface shapes. Two types of aggregates were used in similar mixes (Table 1); (a) local river coarse aggregate and sand, and (b) crushed limestone coarse aggregate and river sand. One slab, 14 by 26 by 6 in., was made from each mix. One-half of this slab was cast against plywood, the other half was cast against a steel liner. Fifteen companion cylinders were also made from each mix; five in steel molds, five in tin can molds, and five in paper carton molds.

Slabs were stored in the mix room, stripped at 8 hr, and covered with plastic film to prevent loss of moisture. The cylinders to be tested at 8, 16, and 24 hr were stored in 100 percent relative humidity at 73.4 F and stripped at time of testing. The 3- and 7-day cylinders were stored in the mix room with the slabs, stripped at 24 hr, and covered with plastic to prevent loss of moisture.

Both types of surfaces of each slab and one cylinder from each mix and surface texture were read in an unrestrained condition with the hammer at each time interval. The cylinders were also read while under an axial restraining load.

The fourth series of tests was made to determine if there was any difference between curved and flat surfaces when both were restrained. The mix using river coarse aggregate and sand was the same as in series 3. Four 5- by 5- by 10-in. prisms were cast against plywood so that specimens having flat test surfaces could be restrained (Fig. 3). Four 6- by 12-in. cylinders were cast in steel molds, four in tin can molds, and four

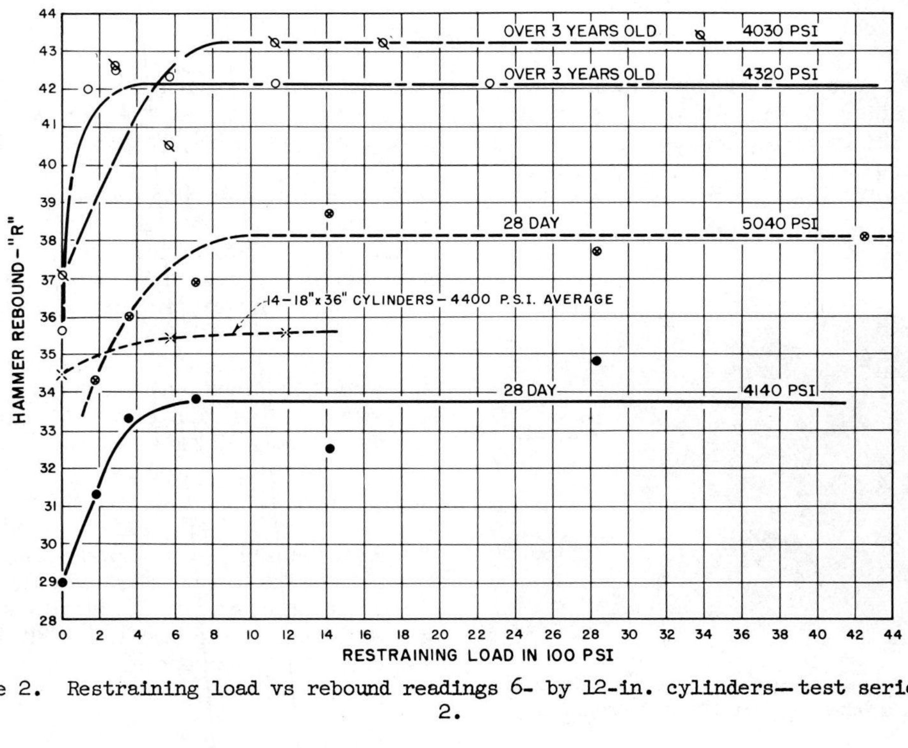

Figure 2. Restraining load vs rebound readings 6- by 12-in. cylinders—test series No. 2.

TABLE 1

CONCRETE PROPERTIES—SERIES 3, 4, AND 5
(Cubic Yard Batch)

Property	Mix No. 1 River Aggregate	Mix No. 2 Crushed Limestone
W/c ratio	0.50	0.52
Water content, lb	261	272
Cement content, lb	519	526
Percent sand	34	38
Slump, in.	3.2	2.7
Percent air	3.0	2.4
Unit weight, pcf	147.3	149.1
Maximum size aggregate, in.	1½ in.	1½ in.

in paper carton molds. The prisms were stored in the mix room, stripped at 8 hr, and covered with plastic. The cylinders to be tested at ages of 8, 16, and 24 hr were stored in 100 percent relative humidity at 73.4 F until they were stripped at time of testing. The 72-hr cylinders were stripped at 24 hr, moved from the fog room to the mix room with the prisms, and covered with plastic to prevent loss of moisture. All specimens were evaluated in both a restrained and unrestrained condition.

Because of the difference in hammer readings for prisms and cylinders, it was thought that the initial curing condition might be affecting the results, so the fifth test series was conducted to eliminate this difference. This series was identical to the fourth series except all specimens were placed in 100 percent relative humidity at 73.4 F, stripped at 8 hr, and stored in the fog room until time of testing.

The difference in readings between loaded and unloaded specimens (Fig. 2) raised the question as to whether this could be caused by the stress condition or be simply a question of effective mass or restraint. This led to the testing under load of 14 heavy 18- by 36-in. cylinders containing 6-in. maximum size aggregate.

DISCUSSION

The purpose of this investigation was to evaluate the rebound readings obtained with the hammer on miscellaneous specimens and on specially prepared specimens by comparing the indicated compressive strength obtained from these readings with compressive strength results obtained by conventional test methods.

Since the instructions furnished with the test hammer recommend the best 10 out of 15 readings to determine "R" and N. G. Zoldners (1) recommends the best 9 out of 15 readings, a calculation was made to determine any appreciable difference between the two methods which might affect the results of this investigation. Information furnished with the test hammer states that the mean value of "R" can be assumed to be reliable when 10 readings of the 15 deviate not more than ±2.5 with an "R" of 15, ±3 with an "R" of 30 and ±3.5 with an "R" of 45. The principal difference between the two methods seems to be that the manufacturers require only 10 reliable readings while Zoldners recommends the use of the middle 9 of 15 reliable readings. Only 15 readings were taken on each specimen; and while the "best 10"

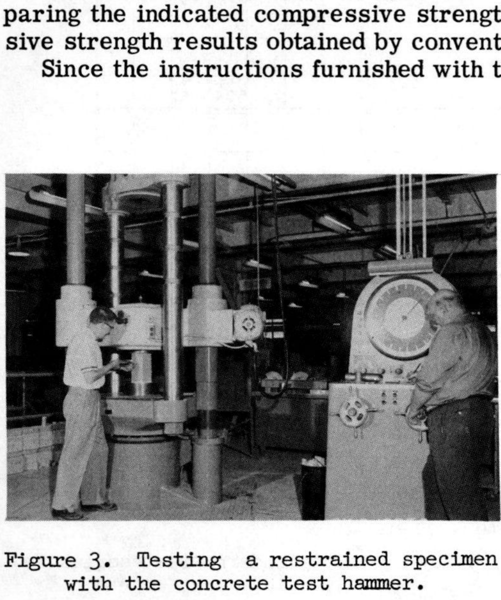

Figure 3. Testing a restrained specimen with the concrete test hammer.

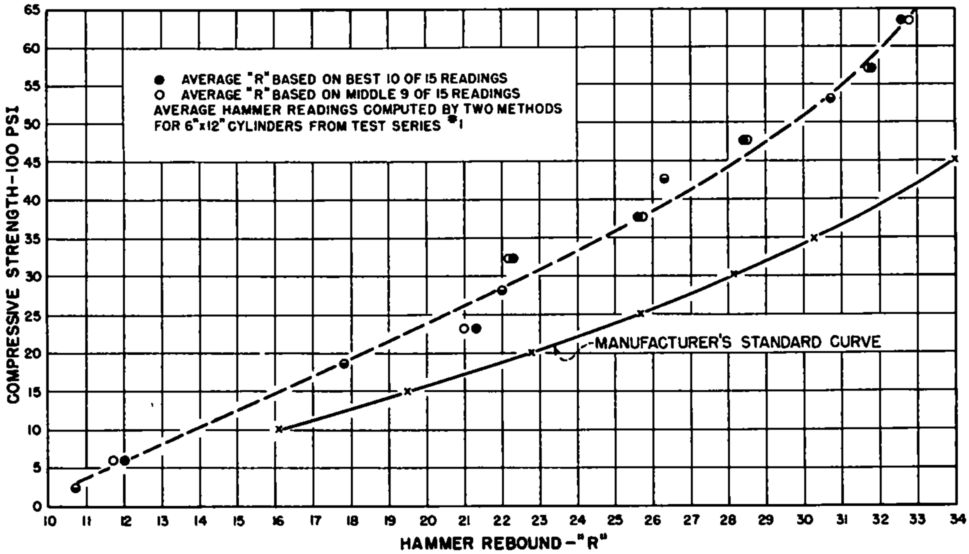

Figure 4. Compressive strength vs hammer rebound readings.

TABLE 2

DEVIATION OF COMPRESSIVE STRENGTH VALUES AT SAME AVERAGE REBOUND READINGS FOR ALL CONCRETE CYLINDERS—TEST SERIES 1, 2, 3, 4, and 5

R	Avg Strength (psi)	No. of Specimens	Standard Deviation (psi)	Coefficient of Variation (%)
10	200	2	40	20.0
11	533	6	140	26.3
12	723	4	179	24.8
13	759	7	212	27.9
14	1,205	4	157	13.0
15	1,103	4	35	3.2
16	1,697	7	526	31.0
17	1,604	7	387	24.1
18	1,833	7	498	27.2
19	2,513	4	509	20.3
20	2,820	15	604	21.4
21	2,885	11	604	20.9
22	3,037	12	713	23.5
23	3,499	17	548	15.7
24	3,554	9	780	21.9
25	3,769	15	519	13.8
26	4,029	16	596	14.8
27	4,045	23	732	18.1
28	4,723	21	642	13.6
29	4,493	17	728	16.2
30	5,075	20	597	11.8
31	4,955	13	1,014	20.5
32	5,579	13	911	16.3
33	5,575	8	495	8.9
34	4,679	2	1,121	24.0
Avg		10	531	18.8

of the 15 readings seldom exceeded the manufacturers' recommended limits, the deviation of all 15 readings was seldom within the proposed limits for reliability.

In determining the best 10 of 15 readings, the 15 readings were averaged and then the 5 readings with the greatest deviation from this average were eliminated. The remaining 10 readings were then averaged to obtain "R". Zoldners' method was modified due to the fact that only 15 readings were taken on each specimen and not all of these 15 readings were within reliable limits. The highest three and the lowest three readings were discarded, and the middle nine averaged to determine "R". These middle nine readings were well within the limits of reliability.

This comparison of methods was made on the first 124 cylinders evaluated in test series 1. While it was found that there may or may not be a slight difference in "R" values for each specimen, the difference is negligible for the average of a number of specimens. These data are shown in Figure 4, and it can be seen that the resulting curve by either method would coincide at the majority of points.

No valid results can be obtained by indiscriminate use of the test hammer. This is shown in Figure 5 where "R" values are plotted against the corresponding compressive strengths for the specimens from test series 1, 3, 4, and 5. The standard deviation of compressive strengths at the same average hammer reading for these specimens fluctuates from 25 lb per sq in. to 1,121 lb per sq in. and the coefficients of variations range from 3 to 31 percent (Table 2).

"R" values for the 18- by 36-in. cylinders are higher within any strength range than for corresponding 6- by 12-in. cylinders (Fig. 5). The methods employed in casting the large cylinders make it improbable that the higher readings are due to striking large aggregate near the surface. Both the 6- by 12-in. and 18- by 36-in. cylinders were evaluated in an unrestrained condition. Since the specimens with the greater weights have the higher readings, it can be assumed that some of the energy of the hammer impact on the smaller specimen displaced the cylinders and resulted in lower rebound readings. When this possible displacement was restricted by a restraining load on the specimens in test series 2, 3, 4, and 5, the "R" values obtained were higher than those obtained on the same specimens in an unrestrained condition (Figs. 2 and 6).

Grieb (2) found that 6- by 12-in. cylinders did not have enough mass or rigidity to give reliable rebound readings unless restrained. However, the rebound values obtained in this investigation on unrestrained cylinders were within the limits of reliability mentioned earlier in the discussion. Further, the standard deviations and coefficients of variation (Table 5) for specimens both unrestrained and effectively restrained are of the same order when the specimens are in the same weight and size category (Fig. 6). Thus, it can be concluded that "R" values determined from the unrestrained condition are no less valid than those obtained in the restrained condition. However, "R" values determined from different conditions or different weight and size specimens cannot be compared. From these facts, it is evident that structural mass might even affect results in field applications.

The wide deviation in strength for the same "R" values (Table 2) can be narrowed considerably by segregating the different specimens according to common factors such as age, aggregate, size, surface, etc. This is shown in Figure 5 and given in Tables 2 and 3 where the average standard deviation is reduced from 531 lb per sq in. to 106 lb per sq in. and the average coefficient of variation is reduced from 18.8 to 9.6 percent by separating the specimens according to aggregate only.

As the restraining load on a specimen increases, the average rebound reading also increases until a maximum is reached, after which an increase in load does not appreciably affect the rebound value (Fig. 2). The restraining load at which the "R" value remains constant appears to vary with the individual specimen; however, from these tests, the effective restraining load for consistent results appears to be about 15 percent of the breaking strength of the specimen. This does not correlate closely with the 250-lb per sq in. effective restraining load indicated by Green (3) or with the 300-lb per sq in. effective restraining load indicated by Grieb (2). Note the inconsistency in the relationship of rebound reading to compressive strength for the specimen shown in Figure 2.

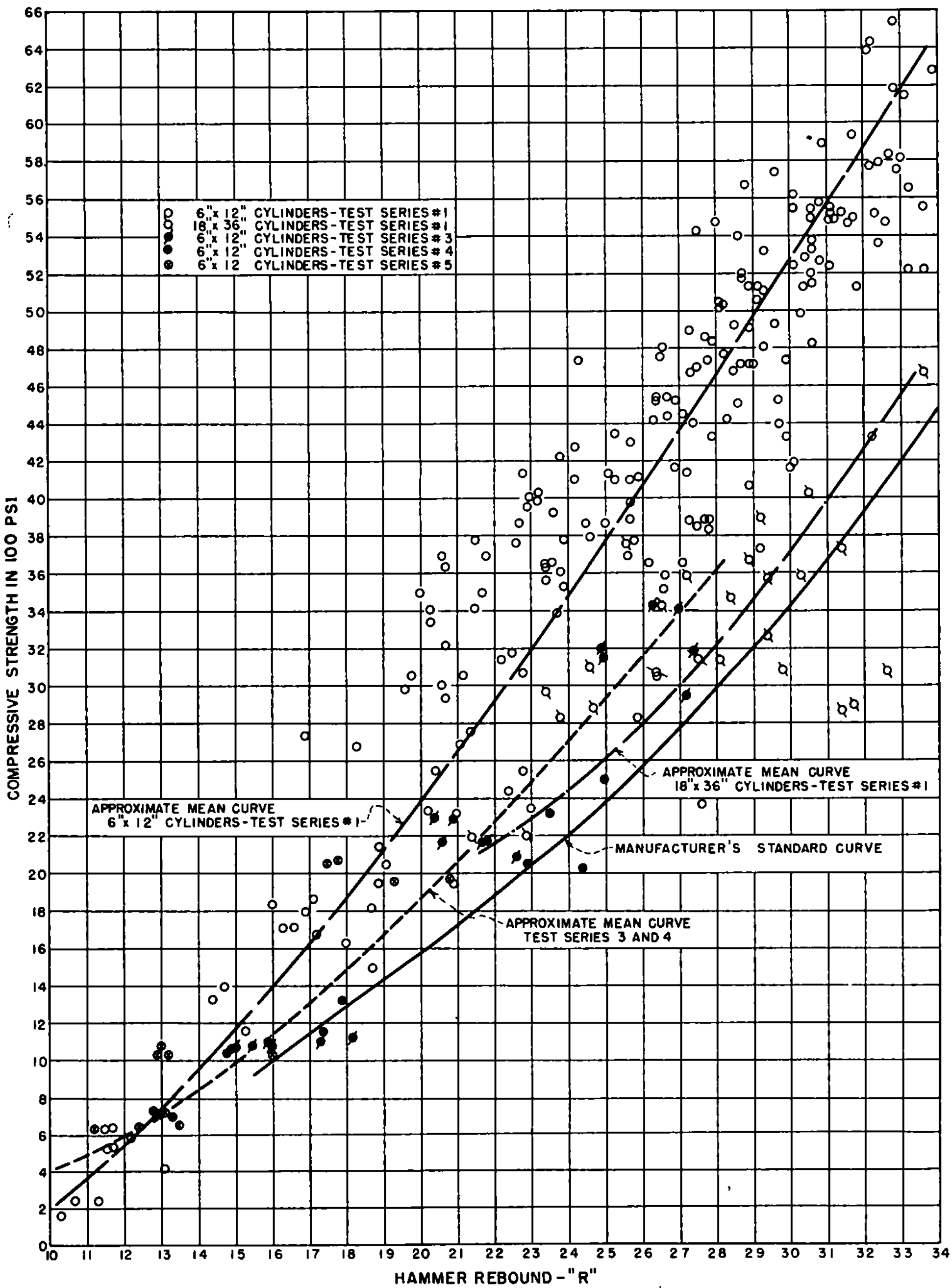

Figure 5. Compressive strength vs rebound reading for unrestrained cylinders.

The restrained 18- by 36-in. specimens exhibited the same tendency to give higher "R" readings under load, but to a lesser degree than did the 6- by 12-in. cylinders (Fig. 2). These data clearly indicate that the "R" reading is a function of the size and rigidity of the test mass. It is probable that the stress condition contributes slightly toward the higher readings in restrained specimens. The size of unsupported areas of a thin structure or the backfilled condition of field structures would probably make a significant difference in the readings obtained.

The impact hammer should be specially calibrated for the conditions of field use, including the size and type structure, aggregate source, mix proportions, and concrete age.

It was determined from the third test series that the rebound readings are affected by the types of aggregate in the concrete. This series showed "R" values for the concrete containing local river aggregate were consistently higher than those for the specimens containing crushed limestone aggregate (Fig. 8).

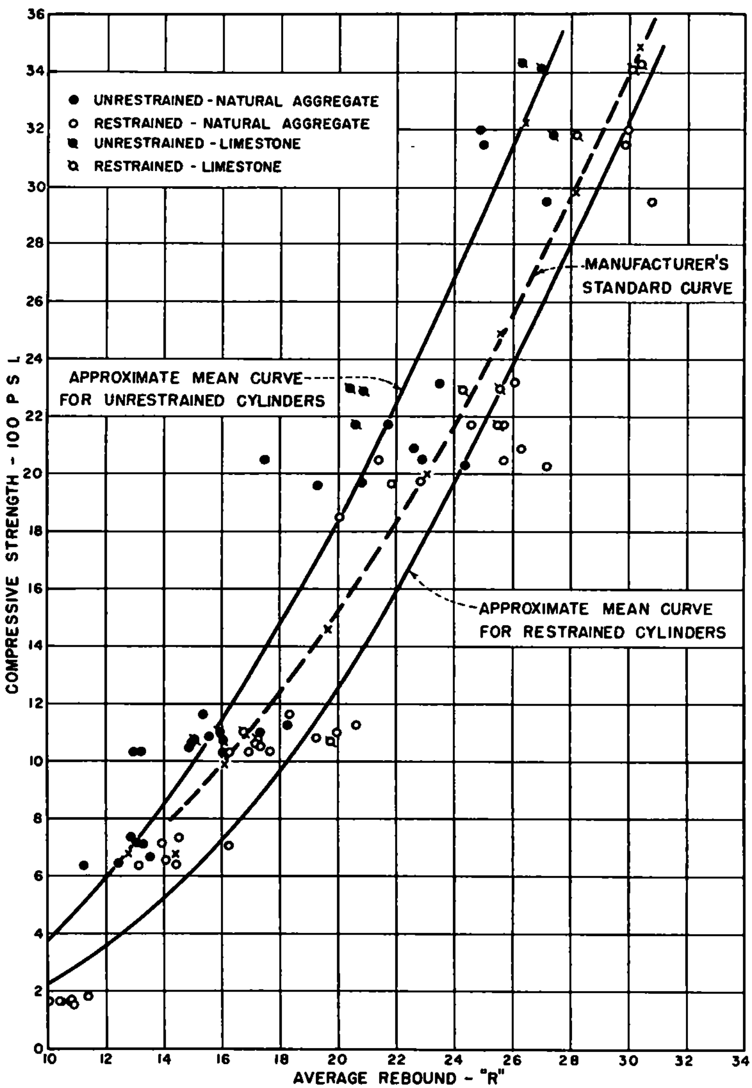

Figure 6. Comparison of rebound readings for 6- by 12-in. cylinders with and without restraining load—test series No. 3, 4, and 5.

TABLE 3

DEVIATION OF COMPRESSIVE STRENGTH VALUES AT SAME AVERAGE REBOUND READINGS FOR ALL CONCRETE SPECIMENS WITH LOCAL RIVER AGGREGATE—TEST SERIES 3, 4, AND 5

R	Avg Strength (psi)	No. of Specimens	Standard Deviation (psi)	Coefficient of Variation (%)
10	169	3	5	3.0
11	183	3	6	3.3
12	245	3	99	40.4
13	700	4	36	5.1
14	689	4	38	5.5
15	720	1	-	-
16	922	3	152	16.5
17	1,046	3	12	1.1
18	1,160	1	-	-
19	1,028	5	155	15.1
20	1,014	4	168	16.6
21	2,005	2	45	2.2
22	1,645	2	325	19.8
23	-	0	-	-
24	2,121	2	51	2.4
25	2,123	3	51	2.4
26	2,183	3	97	4.4
27	2,029	1	-	-
28	-	0	-	-
29	2,824	2	324	11.5
30	3,076	2	125	4.1
31	3,100	1	-	-
32	3,100	1	-	-
Avg		3	106	9.6

Results from the third, fourth, and fifth test series show that flat surfaces give higher hammer readings than cylindrical surfaces (Fig. 7). Companion cylinders cast in steel, tin can, and paper carton molds showed no significant difference between the steel-molded and tin can-molded specimen, but the paper-molded specimens gave higher readings (Fig. 8). This was true even though the steel-molded and tin-molded specimens had a smoother surface and might indicate that the paper form withdraws moisture from the concrete, thus lowering the water-cement ratio at the surface and resulting in a higher strength in this area. Since the hammer primarily tests the surface, it could be possible for the hammer to reflect a nonexistent high strength from a hardened surface.

The third, fourth, and fifth test series showed that the test hammer has no value in testing concrete at very early ages because the hammer rebounds were not great enough to be read accurately on the scale, and further, that the hammer severely scarred the concrete, thus prohibiting its use on green concrete anywhere that it might be exposed to view (Fig. 9). Surface texture causes little significant difference in "R" values at early ages (Fig. 10). This is probably due to the fact that the concrete is still so soft that any difference due to texture is overshadowed by the effect caused by the crushing and displacing action of the hammer on green concrete.

A check was made to correlate "R" with the modulus of elasticity of the concrete specimens tested in series 1. As shown in Figure 11, no valid correlation can be made directly between "R" and elasticity. However, a satisfactory relationship between "R" and elasticity might be obtained if the hammer were to be calibrated for each individual mix tested. Further tests would be required to draw any valid conclusions, and the value of this information is questionable in relation to its applicability and to the expense of deriving it.

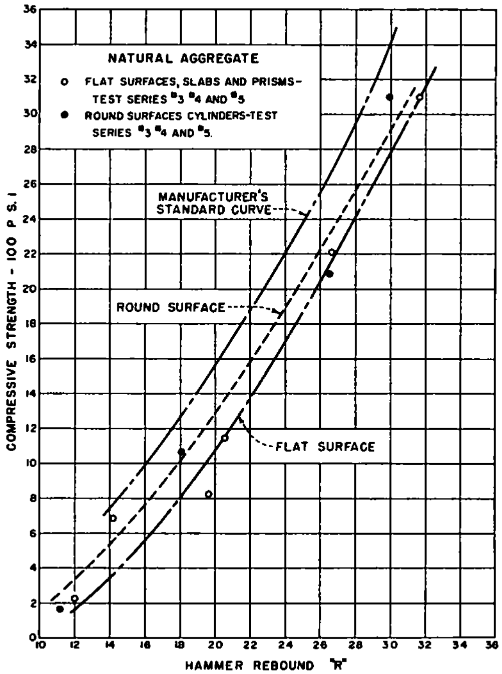

Figure 7. Comparison of surfaces.

Investigations by the Corps of Engineers (4) were extensive enough to conclude that hammer readings taken on dry concrete surfaces will be generally higher than readings taken on wet surfaces, and that hammer readings taken with the hammer held in a horizontal position are generally higher than those obtained with the hammer in a vertical position.

From observations in this and other previously published investigations, it appears that the impact-type concrete test hammer gives a correlation between compressive strength of concrete and rebound values. However, indiscriminate use of the hammer will give misleading results. The deviation in strengths indicated by any rebound value can be narrowed from wide limits to reasonable limits by calibration of the test hammer. A calibration should be made for each mix being used on a job under both wet and dry surface conditions and with the hammer both vertical and horizontal. Correction factors should be derived to compensate for use of the

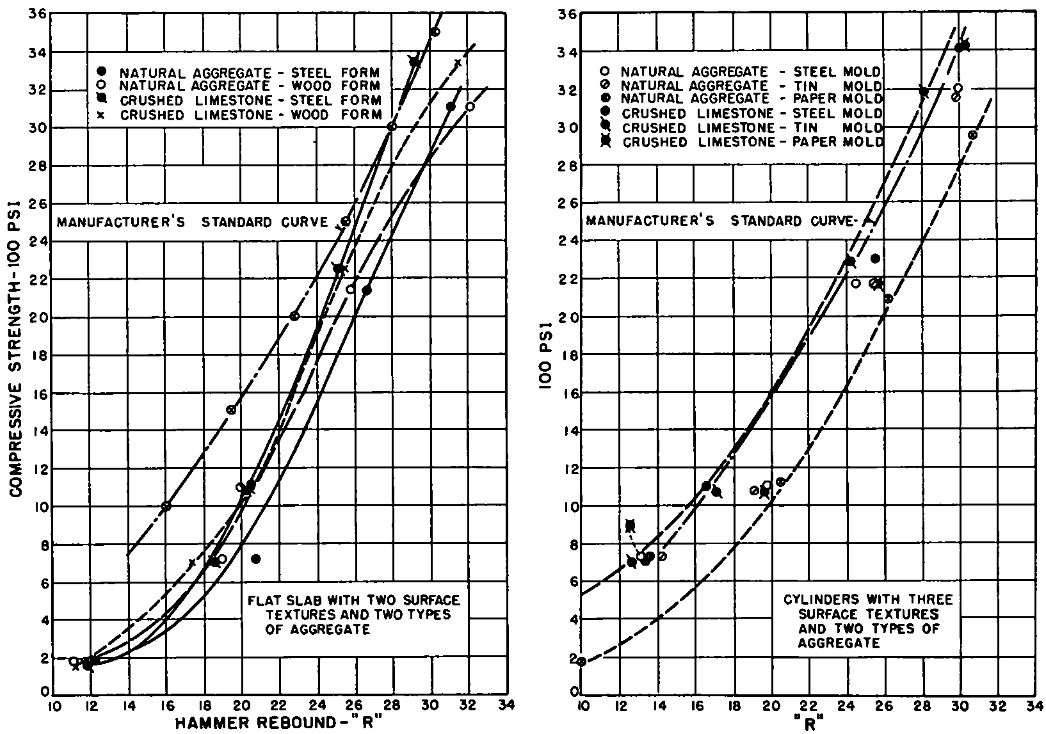

Figure 8. Comparison of shapes, surface textures and aggregates— test series No. 3.

TABLE 4

DEVIATION OF AVERAGE OF REBOUND READINGS WITHIN NARROW STRENGTH RANGES FOR CONCRETE CYLINDERS–TEST SERIES 1

Strength Range 100 Psi	No. of Specimens	Avg, R	Standard Deviation	Coefficient of Variation
0- 5	7	10.7	1.1	10.3
5-10	5	11.7	0.2	1.7
10-15	3	15.9	2.0	12.6
16-18	4	17.0	1.3	7.6
18-20	6	18.1	1.6	8.8
20-22	4	20.6	1.7	8.3
22-24	4	23.0	2.9	12.6
24-26	3	24.8	3.3	13.3
26-28	4	19.4	1.9	9.8
28-30	8	25.2	4.2	16.7
30-32	12	24.8	3.9	15.7
32-34	4	23.5	3.6	15.3
34-36	14	25.7	4.1	16.0
36-38	17	24.5	2.8	11.4
38-40	15	25.7	2.0	7.8
40-42	14	26.3	2.6	9.9
42-44	9	27.0	2.6	9.6
44-46	12	27.5	1.1	4.0
46-48	12	28.4	2.1	7.4
48-50	10	28.5	1.2	4.2
50-52	12	29.2	1.1	3.8
52-54	12	31.0	1.4	4.5
54-56	17	30.8	1.5	4.9
56-58	6	31.1	1.7	5.5
58-60	6	32.6	1.1	3.4
60-65	6	32.8	0.6	1.8

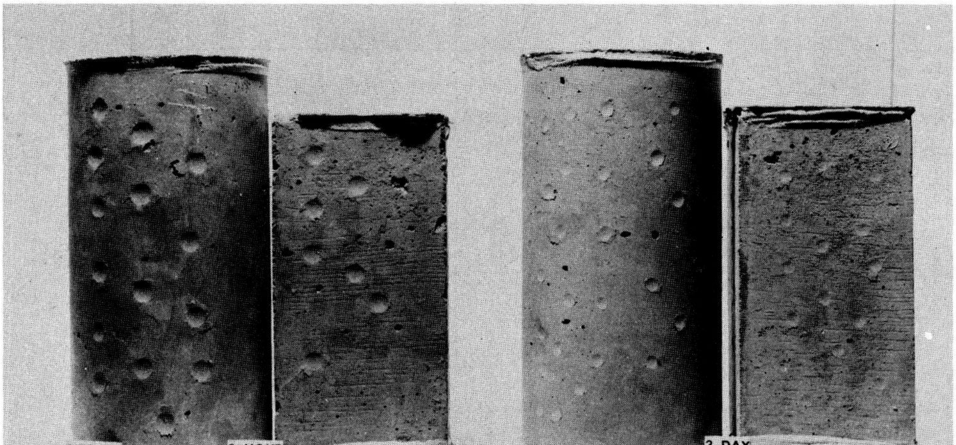

Figure 9. Early age specimens showing pocking due to concrete test hammer impact. Eight-hour specimens on left, and 3-day specimens on right.

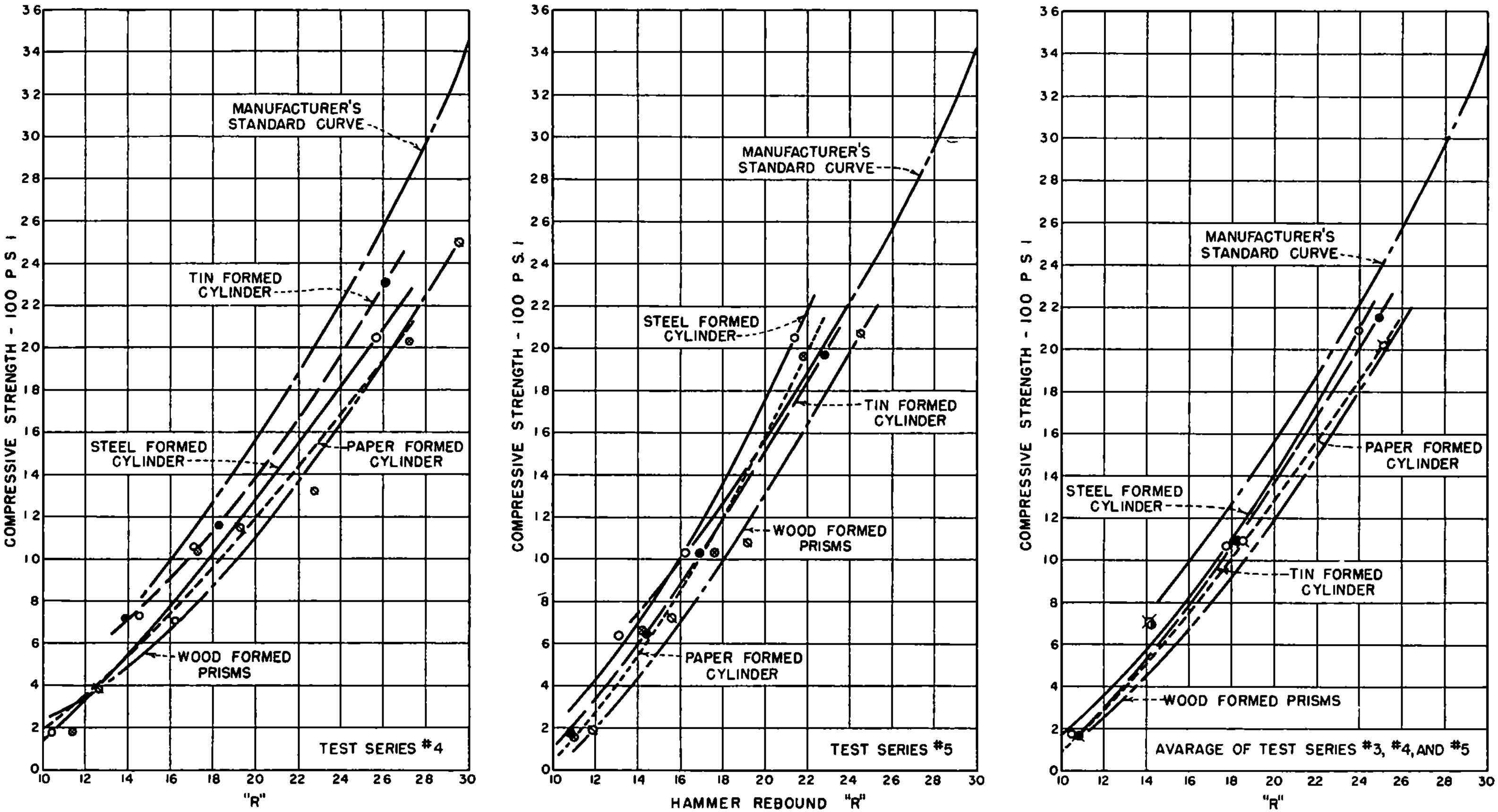

Figure 10. Comparison of surface textures and shapes— test series, 3, 4 and 5— natural aggregate.

TABLE 5

DEVIATIONS OF COMPRESSIVE STRENGTH TESTS AND HAMMER REBOUND READINGS ON RESTRAINED AND UNRESTRAINED 6- BY 12-IN CYLINDERS—TEST SERIES 3, 4, AND 5

		Compressive Strength			Unrestrained Condition			Restrained Condition		
No. of Cylinders	Age (hr)	Psi	Standard Deviation (psi)	Coefficient of Variation (%)	Avg, R	Standard Deviation	Coefficient of Variation	Avg, R	Standard Deviation	Coefficient of Variation
5	8	172	6	3.5	Too low to be read	-	-	10.7	0.4	3.7
6	16	682	36	5 3	12 7	0 8	6.3	14 4	0 9	6.3
12	24	1,076	39	3.6	15 4	1 4	9 1	18.1	1 4	7.7
12	72	2,131	120	5.6	21 4	1.8	8.4	24.8	1.8	7.3
6	7-day	2,221	163	5.1	26.3	1.0	3 8	29.9	0.8	2.7

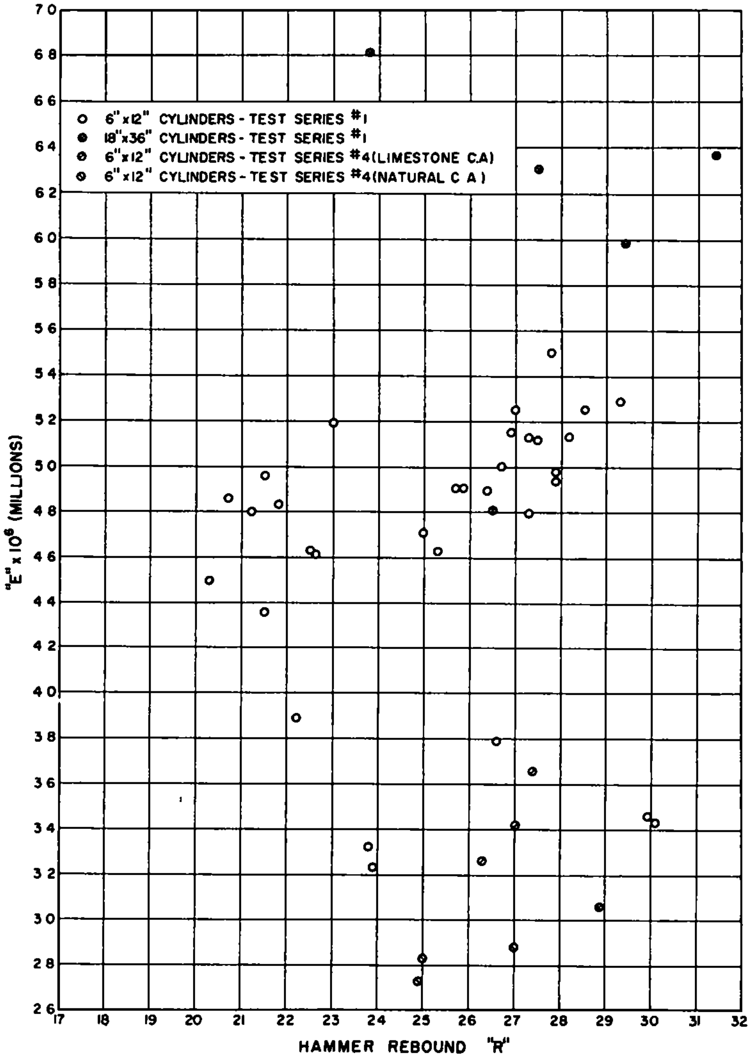

Figure 11. Comparison of hammer readings and modulus of elasticity.

hammer on other than flat surfaces, for use of the hammer at angles other than horizontal and vertical, and to compensate for deviations due to surface textures from forming materials other than those used for the original calibrations. The curves furnished by the manufacturer should not be used. Grieb found that the manufacturer's curve was conservative in practically every instance which the investigation verifies so long as the specimen is small and young or unrestrained. However, the data shown in Figures 2 and 6 indicate that for either old concrete, heavy specimens, or restrained specimens, the reverse is likely to be true.

At this time, there have been no investigations involving the use of the test hammer on reinforced concrete. It is likely that very heavily reinforced concrete will cause erratic hammer readings which would preclude its use for testing in this type of construction.

The test hammer, when calibrated properly, could be an effective aid to field testing of concrete, but no amount of calibration will be sufficient for it to replace the conventional test methods.

The expense of calibration should be weighed against its value as a simple and rapid check for concrete quality. Above all, its limitations and its proper use should be understood by all concerned prior to its acceptance as a testing tool.

CONCLUSIONS

1. A usable relationship exists between readings (R) obtained from the impact-type concrete test hammer and the compressive strength of concrete (Tables 2, 3, 4, and Figs. 2, 4, and 5). This relationship will be closer if special calibration curves are provided for each particular application.
2. The test hammer is not suitable for either very early age tests or where concrete strength is less than 1,000 lb per sq in., (Fig. 9).
3. Different surface shape, texture, aggregate types, condition of cure, or moisture content cause measurable variation in rebound readings.
4. Rebound readings increase with restraining loads up to about 15 percent of specimen strength, indicating that the hammer readings are a function of the size or rigidity of the test mass (Fig. 2).
5. The use of a test hammer on concrete specimens selected at random is not reliable due to the extreme variations of strengths obtained from concretes having the same "R" value (Tables 2 and 4 and Fig. 5).
6. Other factors being equal, flat surfaces produce higher hammer readings than rounded surfaces (Figs. 7 and 10).
7. The "R" value cannot be directly correlated to the modulus of elasticity of concrete (Fig. 11).

REFERENCES

1. Zoldners, N.G., "Calibration and Use of Impact Test Hammer." ACI Journal Proc., Vol. 54, p. 161 (Aug. 1957).
2. Grieb, William E., "Use of the Swiss Hammer for Estimating the Compressive Strength of Hardened Concrete." Public Roads 30:2, p. 45 (June 1958).
3. Green Gordon W., "Test Hammer Provides New Method of Evaluating Hardened Concrete." Also a discussion by six authors, ACI Journal Proc., Vol. 51, p. 249 (Nov. 1954).
4. "Investigation of the Schmidt Concrete Test Hammer." U.S. Army Engineer Waterways Experiment Station, Miscellaneous Paper No. 6-267 (June 1958).

Discussion

W.H. CAMPEN, Omaha Testing Laboratories—Although the test hammer is not an accurate instrument for determining the compressive strength of concrete, it is a fine qualitative instrument. As such it can be used for a number of purposes. I wish to mention two cases in which it proved very useful.

One case involved a large number of pedestals in an electrical sub-station. Due to cracking and spalling when the superstructures were being placed, the concrete in the pedestals was questioned by the engineer. The writer was engaged to investigate. He eventually tested all the pedestals with the hammer and classified the strengths as good, doubtful, and poor. Cores were then taken from the representative groups and tested for strength and cement content. The results confirmed the indications of the hammer.

Another case involved an exposed floor in a power plant. Soon after the floor was poured, a cold wave came along and although the floor had been covered and provided with heat, parts of it failed to set properly. The hammer identified the parts which had set properly as well as those which had not. Eventually, during additional curing, the hammer was used to indicate when the concrete in all of the floor attained uniform strength.

Tests for Freeze-Thaw Durability of Concrete Aggregates

BAILEY TREMPER and D. L. SPELLMAN, respectively, Supervising Materials and Research Engineer and Senior Materials and Research Engineer, California Division of Highways, Sacramento

Tests of aggregate in air-entrained concrete have been made by methods suggested by T. C. Powers (1) for resistance to freezing and thawing. The procedure differs from that of currently used test methods in several important respects. Among these are (a) maintenance of the original moisture in the aggregates, (b) testing of the largest particle sizes to be used in the work, (c) subsequent conditioning of the cured concrete by drying to a degree found appropriate to exposure conditions at the site of construction, and (d) freezing at a rate commensurate with natural conditions. Methods and apparatus used in conducting the tests are described, and results of variations in test procedure are shown.

Specifications based on the test procedure have been used for the acceptance of aggregates in construction work that is subject to severe winter conditions at high elevations in California. Many of the aggregates would not be considered to be acceptable under commonly used freeze-thaw methods. One hundred and seventy-three lane-miles of pavement have been subjected to one or two winters of severe exposure. At present, the concrete is judged to have withstood the effects of exposure without evidence of distress due to freezing and thawing.

●THIS REPORT describes test methods used to evaluate the frost resistance of aggregates when incorporated in air-entrained concrete. The concept of the test procedure was provided by Powers (1, 2). As far as known, the acceptance under contract procedure of aggregates based on this concept has not previously been undertaken.

In California most of the concrete pavements have been constructed at relatively low elevations where freezing weather is of rare occurrence. Until recently, the Division of Highways has had little occasion to study the ability of locally available aggregates to produce frost resistant, air-entrained concrete.

A decision to pave 70 mi of Rt 80 between Colfax at an elevation of 2,500 ft and the Nevada state line near Reno with portland cement concrete, provided the impetus for the extensive investigation of available sources of aggregates. This road reaches an elevation of 7,135 ft at Donner Pass, then drops to an elevation of 5,000 ft at the Nevada state line. Precipitation is heavy on the western slope. The annual snowfall near the summit is among the heaviest in the United States. Temperatures as low as -25 F are not uncommon near the Nevada state line.

In considering methods of making freezing and thawing tests of concrete, the two papers by Powers (1, 2) were studied carefully. They were believed to contain a number of proposals of distinct merit. Equipment to perform tests by the Powers procedure as well as that for making the four ASTM tests was obtained.

Six points made by Powers are considered to be of importance. These are given as follows, along with the test procedure used to put them into effect.

SIZE OF AGGREGATE

There is a critical size of aggregate with respect to its frost resistance. In general, the larger its size the greater its probability of being vulnerable. Therefore, test specimens should contain the largest size of aggregate to be used in the work and the dimensions of the specimen should be adequate for this purpose.

Specifications for the projects under consideration required that the concrete contain 1½-in. maximum size aggregate. This size range was used in preparing laboratory mixtures for test. Cylinders 4½ in. in diameter by 9 in. high were molded for the Powers test. While it might have been possible to consolidate specimens of smaller size, the number of the large size particles of aggregate in each specimen would have been reduced. Since the larger particles are more likely to be the critical ones, it was not thought advisable to use a smaller test specimen. Specimens for ASTM rapid freezing and thawing in water were 4- by 5- by 18-in. prisms, on which dynamic E was measured, or 4½- by 9-in. cylinders, on which length changes were measured.

MOISTURE IN AGGREGATES

Aggregates that have been dug from locations below the water table if subsequently allowed to dry may not regain their full amount of water by simple soaking for a reasonable length of time. If the aggregates as incorporated in test concrete are not saturated to a degree comparable to the condition in which they are used in the work, the test results can be very misleading.

Specifications require that aggregates be washed before use. In practice, aggregates are usually dug, screened, washed, and batched without any opportunity for drying. Preliminary test samples of pit run material were taken below the water table. Samples from manufactured stocks were taken only where free surface moisture was visible. They were placed in metal cans with tight fitting covers. Additional water was placed in each can. In the laboratory, the aggregates were maintained in a thoroughly wet condition during screening and other processing and were introduced into the mixer while wet. It might have been possible to resaturate dried aggregates under vacuum but uncertainty as to completeness of saturation led to the adoption of the first procedure.

AIR BUBBLE SPACING

For tests of air-entrained concrete, the paste should be protected with bubbles, and adequate protection requires that the calculated spacing factor not exceed 0.01 in.

The laboratory is not equipped to make linear traverse measurements of polished sections. It was assumed that the use of neutralized Vinsol resin in an amount to result in a measured air content of 4.5 ± 0.5 percent would fulfill the bubble spacing requirement.

RATE OF COOLING

The rate of cooling in the laboratory test should not be greatly higher than the rate experienced under natural conditions of exposure. The use of high cooling rates in the laboratory as required in some current methods subjects the concrete to internal hydraulic pressures of a magnitude much greater than experienced in nature and may produce misleading results. Such a test may serve to reject aggregates that would perform satisfactorily in service.

Prior to starting tests, an experimental slab 12 ft square had been installed with thermocouples at Donner Pass, the highest elevation of the proposed construction. Temperature measurements were recorded during one winter. In the range below 32 F, the greatest rate of temperature drop within the concrete did not exceed 3 F per hour except on one day when a drop of 6 F in one hour was recorded at a point just below the surface of the slab. Powers has suggested a cooling rate in the test of 5 F per hour and this rate was adopted in the work.

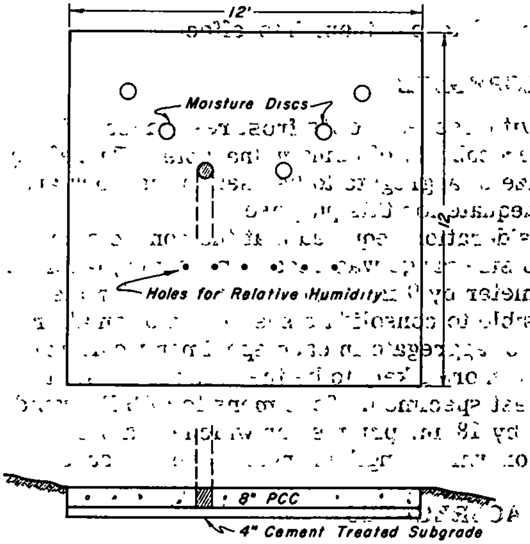

Figure 1. Typical test slab for moisture measurement.

MOISTURE IN CONCRETE

Unless the concrete is to be exposed in such a way that it will never have an opportunity to dry, it will dry to some extent during each summer season. Concrete specimens should be conditioned by partial drying to a degree comparable to field conditions before subjecting them to freezing and thawing.

The four ASTM methods require that freezing and thawing tests be started after 24 hr of moist curing in the fog room followed by immersion in water saturated with lime to the age of test. Corps of Engineers method C 10-54 has a similar requirement except that storage is in the fog room until 48 ± 4 hr prior to test when they are stored in saturated lime water. None of these methods provides for preliminary drying of the concrete.

California Division of Highways has conducted tests of concrete slabs for the purpose of estimating the distribution of moisture in concrete pavements throughout annual cycles of weather. The first of these experiments was conducted in Sacramento during 1953-4. Subsequently test slabs have been installed at two locations at high elevations on Rt 80. Although the slabs have differed somewhat in design detail, a typical layout is shown in Figure 1.

The test slabs are 12 ft square and 8 in. thick. They are supported by a cement-treated subgrade with bituminous seal in accordance with prevailing practice in California. The slabs are located in driveways leading to maintenance stations. These sites were selected to provide assurance that snow would be removed at the frequency occurring on the highway proper.

Holes 6 in. in diameter extending from the surface to the subgrade, were formed ahead of placing the concrete. During placing, a number of 6- by 12-in. cylinders were molded. A central rod in the mold provided a cast-in-place hole at the longitudinal axis of the cylinder. After curing for 14 days, the cylinders were sawed into discs 1-in. thick which were then lapped to provide intimate contact between them when stacked. Each disc while still in an undried condition, was weighed to the nearest gram and then subjected to drying at 220 F to 230 F to essentially constant weight. The loss in oven drying is called the "evaporable water." The value is not constant between discs because of non-uniformity in distribution of coarse aggregate in specimens of such size. Typically, each disc contained about 50 g of evaporable water. A change of 1 g in weight therefore, represented a change of about 2 percent in evaporable water content.

After drying, the discs were soaked in water for several days and then bolted together to form a stack 8 in. high as shown in Figure 2. The assemblies were then inserted in the precast holes of the test slab. A small amount of calking compound was placed in the annular space at the surface to prevent access of surface water. The diameter of the discs was about 0.02 in. less than that of the hole. The clearance was so small that considerable practice was required in assembling the stacks so that they could be inserted in the slab. Regardless of the moisture content of the discs relative to that of the slab at the time of insertion, it is believed that equilibrium with the slab proper becomes established after a period of time.

Test slabs were constructed in August. Discs were inserted in September and first removed for weighing in October. They were weighed subsequently at monthly intervals up to some date in late November or early December when the slab became frozen to its entire depth. Periodic weighings were resumed in the spring and continued until freezing weather the following winter. Changes in weight of each disc were recorded as percentages of evaporable water.

Each of the test slabs contained a series of metal-lined holes with the concrete at the bottom exposed at varying depths. Metal plugs remained in place at the top of the sleeves except when electric hydrometers were inserted to measure the relative humidity of the air within the concrete.

Thermocouples were installed at varying depths within the slab and temperatures were recorded automatically during the winter season.

The first of such test slabs, containing only moisture discs, was installed in a field at Sacramento (elevation 25 ft) in August, 1953. Actually there were two smaller slabs each containing two stacks of discs. One slab was on a cement-treated subgrade with bituminous seal, the other on natural earth. The presence or absence of a cement-treated subgrade with bituminous seal made no significant difference in the measured moisture changes in the slabs during the ensuing year. Figure 3 shows the moisture changes during an annual cycle at Sacramento as the average of changes in the four stacks of moisture discs. It will be noted that the top disc, 1 in. in thickness, became progressively drier starting in March, and by September contained only 26 percent of its evaporable water. Changes in the lower discs were progressively less. The bottom disc lost only 8 percent of its evaporable water despite the fact that less than 0.2 in. of rain fell between the middle of May and the first of November. Summer weather at Sacramento is hot and the relative humidity is low.

Figure 4 shows moisture changes at Donner Summit (elevation 7,135 ft) as the slab approached two winters. Changes in the top set of discs are shown independently. The balance are grouped within the band as shown to avoid confusion. The significant fact to be noted is that although the top disc responded to short changes in weather conditions, the balance of the concrete slab entered the period of severe winter freezing in 1956 with an evaporable water content of 85 to 89 percent. The slab contained slightly more moisture in December-1957.

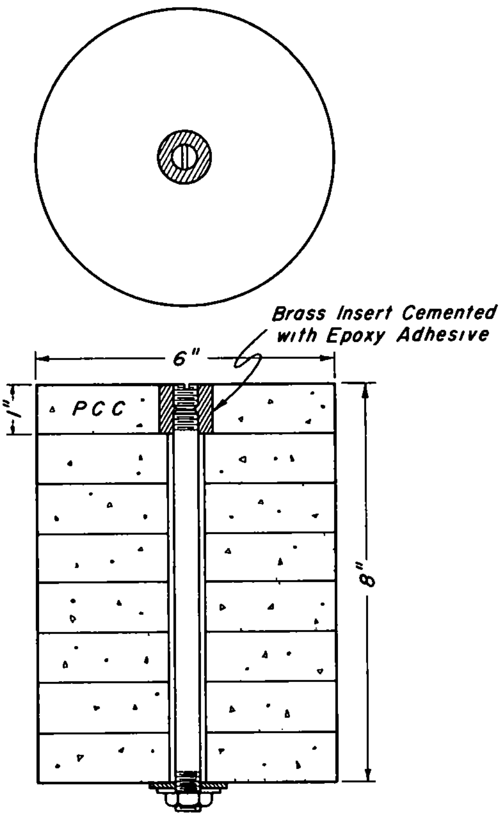

Figure 2. Assembly of moisture discs.

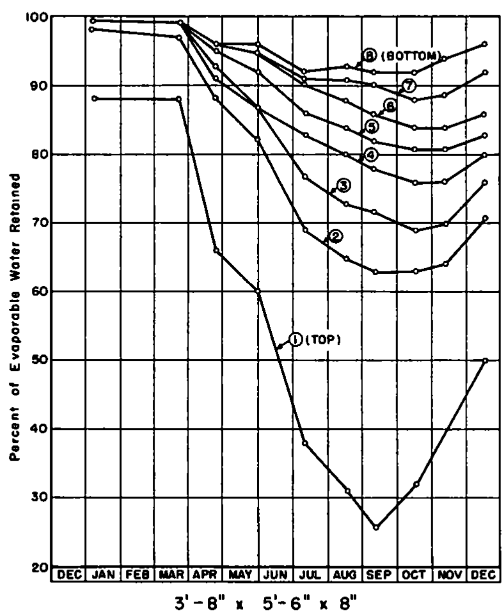

Figure 3. Test slabs at Sacramento (placed Nov., 1953).

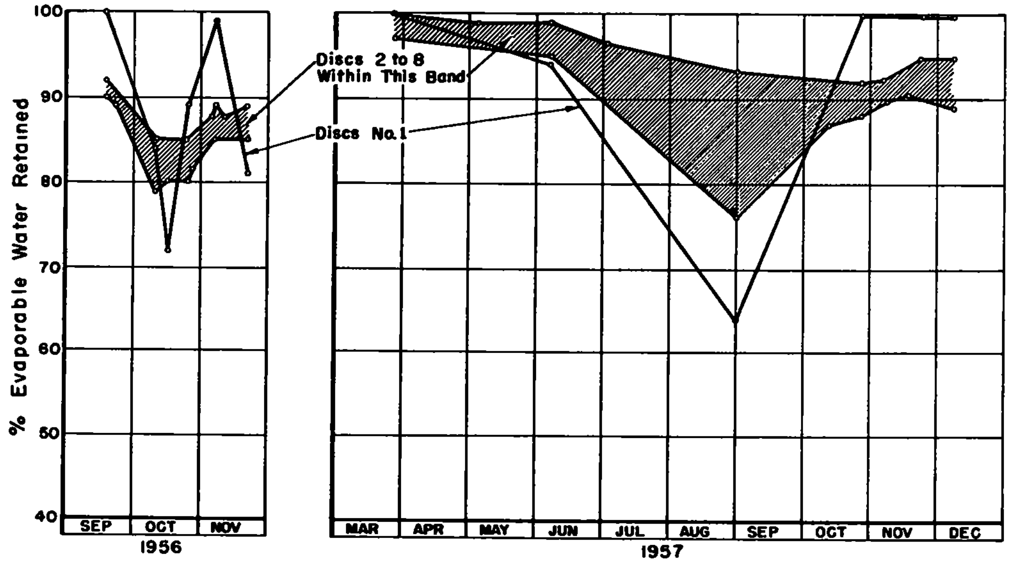

Figure 4. Test slab at Donner Summit (placed Aug. 9, 1956).

A third test slab at Yuba Gap (elevation 5,700 ft) yielded results similar to those at Donner Pass, except that a higher degree of saturation resulted. The site, however, was at a poorly drained location which is not considered to be representative of a modern highway. The results are not given in this report.

Measurements of relative humidity within the slabs were not completely reliable particularly when the temperature was below 45 F. However, it is believed that relative humidities (given in Table 1) at the start of the winter season are reasonably correct.

From the data obtained, it was concluded that specimens containing the kind of concrete present in the test slabs would be suitably conditioned for freeze-thaw tests if, after fog-room curing for 14 days, they were allowed to dry until the evaporable water content reached about 85 percent and the relative humidity was also about 85 percent. Experiments with test cylinders containing aggregates similar to those of the Donner Pass test slab indicated that they lost 15 percent of evaporable water when subjected to drying at room temperature and about 50 percent relative humidity for 48 hr. It was expected that concrete containing other aggregates might lose water at different rates. Also, it was believed that the best representation of site conditions would be obtained by a constant period of drying rather than by drying to a predetermined loss in weight. Therefore, the time of conditioning test specimens by drying was established at 48 hr. In order to effect a more uniform distribution of moisture within the specimen and to approach a relative humidity of about 85 percent within the concrete, the air-dried specimens were placed in sealed containers above a saturated solution of sodium acetate for 5 days. The theoretical relative humidity of the atmosphere surrounding the specimens was 76 percent at 68 F. It was assumed that after 5 days, the relative humidity within the specimen was about 85 percent. Subsequent tests, however, have shown that this procedure frequently produced a degree of drying somewhat greater than was intended. Later in this report, data are presented to show the effect of different degrees of drying.

TEST PROCEDURE

The proposed approach is to determine the change in length of concrete while it is being slowly cooled below the normal freezing point. If the concrete shrinks normally in the freezing range, it is immune at the time of test. If it dilates, it is not immune; the process that eventually causes disintegration has begun. It is proposed to make such a test after each two weeks of water-soaking a specimen that previously has been conditioned in air to represent job expectations. Soaking should be continued until the longest safe period has been found. Loss of water from the specimen during freezing should be prevented. If the safe period of soaking exceeds the probable duration of freezing weather each year at a specific construction site, the concrete may be considered to be safe from the danger of damage from freezing at the site.

TABLE 1

RELATIVE HUMIDITY WITHIN TEST SLABS AT START OF SEVERE WINTER WEATHER

Date	Depth Below Surface of Slab (in.)	Relative Humidity (%)
(a) Donner Pass—First Year, 1956		
11/7/56	1	83
	2	85
	3	90
	5	98
	7	89
	9 (in subgrade)	82
11/20/56	Slab temperature below 32 F Readings not reliable	
(b) Yuba Gap—First Year, 1957		
11/22/57	¾	76
	1½	89
	3	74
	5	94
	7	98
	8½ (in subgrade)	89
12/12/57	Slab temperature below 32 F Readings not reliable	

Specimens were molded from concrete containing 5½ sacks of cement per cubic yard to which sufficient Vinsol resin solution was added to produce 4½ ± ½ percent air and water to give a slump of 2 in. Originally, duplicate specimens were molded in the form of 4½- by 9-in. cylinders with gage studs and 4- by 5- by 18-in. prisms. The cylinders were tested by the Powers procedure. The prisms were used in rapid freezing and thawing in water (ASTM C 290) on which changes in dynamic E were measured. Later 4½- by 9-in. specimens were used in the rapid freezing and thawing in water test and deterioration was measured by change in length.

Originally each specimen for the Powers procedure contained a thermocouple, but it soom became evident that all specimens in a cooling bath cooled at the same rate. Thereafter, a single dummy specimen with a thermocouple was used for temperature measurement. Prior to testing by the Powers procedure, the specimen was placed in a rack made of Invar steel, except for a brass insert at the top, on which was mounted a linear variable differential transformer (see Figs. 5 and 6).

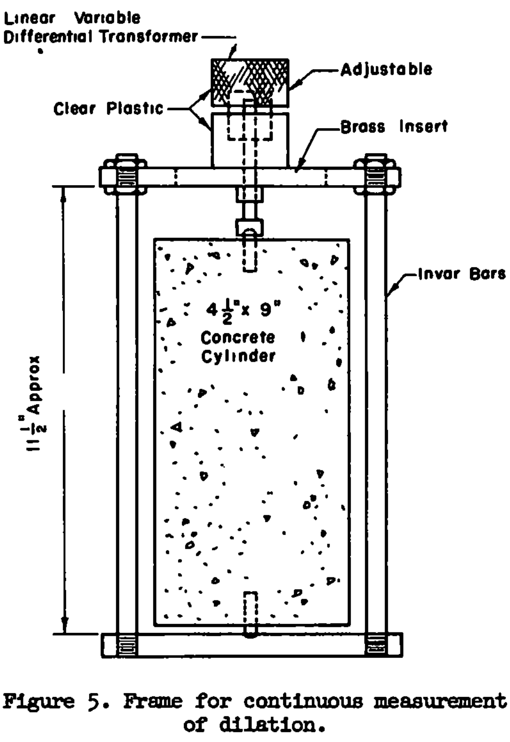

Figure 5. Frame for continuous measurement of dilation.

Length changes indicated by the transformer were recorded continuously on a strip chart. Temperature changes were recorded on another chart. Records were obtained while the specimens were being cooled from about 50 F to 0 F. Equal time intervals were laid off on each chart and a plot of length change versus temperature was constructed for each specimen. The characteristics of the plotted curve were used to evaluate the behavior of the specimen as it was cooled above and below the freezing point.

Cooling was accomplished in a bath of water-saturated kerosene as a means of preventing gain or loss of moisture from the specimen. Cooling equipment consisted of two small commercial household freezing boxes which were lined with copper sheeting to prevent leakage of kerosene to the insulation. The units have inside dimensions of 14 by 26 by 18 in. deep and each provides space for four test specimens plus a dummy specimen containing a thermocouple. The kerosene was agitated by a small circulating pump, which was used only to establish equilibrium conditions at the start of a test. Rate of cooling was adjusted to 5 F per hour quite successfully without automatic control. Views of cooling units in operation are shown in Figures 7 and 8.

INTERPRETATION OF RESULTS

Discussions with other engineers revealed that methods of making freezing and thawing tests, and the establishment of test limits generally have been adjusted in accordance with service experience in the locality. California did not have an extensive service record of air-entrained concrete in severe climates and thus was handicapped with respect to a basis for judgment of locally available materials.

A concrete pavement of non-air-entrained concrete had, however, been constructed at Donner Pass in 1937. This pavement suffered surface scaling early in its history but the concrete otherwise has remained in excellent condition. Concrete aggregates from the American River near sacramento had been used in this construction. In this report, aggregates from this source will be designated as No. 1. It was learned that a considerable number of minor structures had been constructed in Nevada over a period of several years using air-entrained concrete containing aggregates produced from the Truckee River in the vicinity of Reno. Examination of these structures led to the conclusion that this aggregate (No. 2) was capable of producing durable concrete.

Two examples of known durable aggregate which could be obtained for testing purposes were thus provided. Samples of an aggregate of known poor service history—a limestone from the Rapid formation in Iowa—were obtained through the courtesy of the Iowa State Highway Commission.

Results of test with the three aggregates described above provided guide marks in establishing quantitative limits of performance in the Powers procedure.

Figure 9 shows idealized cooling curves. Curve 1 represents thermal contraction above the freezing point. The measured slope is not strictly proportional to the thermal coefficient for two reasons. The first is because the over-all thermal coefficient of the frame supporting the specimen is not zero. The second is because movement of water within the paste and aggregate does not have time to reach complete equilibrium when the temperature is being lowered continually. Curve 2 represents the contraction after ice begins to form in the concrete. Ice crystals under progressive cooling, tend to attract moisture at the expense of that in the paste, causing the latter to shrink at a rate greater than that due to thermal contraction alone. The point of intersection of Curves 1 and 2 indicates the temperature at which ice begins to form. If experimental curves could be obtained with the precision of those shown in Figure 9, the freezing point could be determined accurately.

Curve 3 represents the type of result that has been obtained in certain instances. In this case, there is little or no change in length while the specimen is being cooled several degrees below its freezing point. Eventually the curve resumes a downward slope. Dilation has occurred as measured by the distance, a, which is the greatest distance between Curves 3 and 2. Dilation of this type is extremely difficult to measure from the plotted curves obtained in the study.

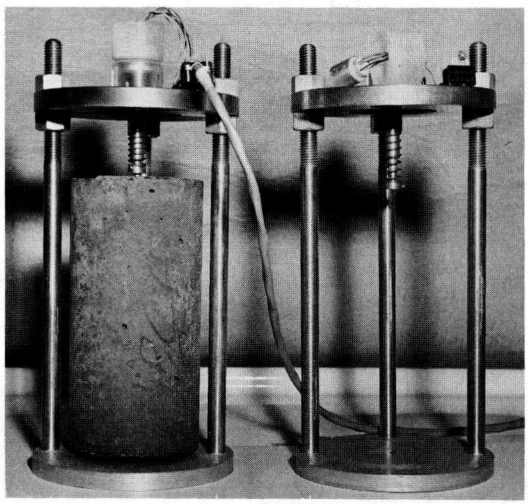

Figure 6. Frames for continuous measurement of dilation.

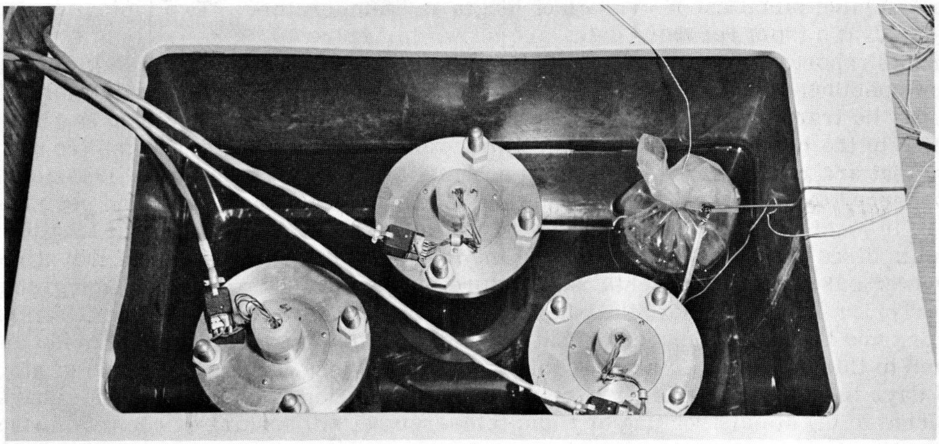

Figure 7. Specimens in cooling unit.

The most common type of curve that is obtained in the work when the concrete is not immune, is represented by Curve 4. Here there is an abrupt expansion at the freezing point. The curve rounds quite sharply and then assumes a downward slope. The distance, b, is easily measured; however, it does not represent the entire dilation because of its nearly horizontal trend over a few degrees of cooling. The distance, b', represents the complete dilation but is difficult to measure in practice.

It is evident that for the purpose of acceptance or rejection, the selected limitation on dilation must be one that can be measured with reasonable assurance. The chart on which length changes are recorded can be estimated to the nearest 0.000025 in. The gage length of the test specimen is 7½ in., therefore, the recorded length change can be estimated to the nearest 0.000003 in. per in. The chart on which temperatures are recorded can be read to the nearest 1 F. This is the temperature near the center of the specimen and the outside is slightly cooler. Although separate charts are used for recording length and temperature, it is believed that there is no significant error

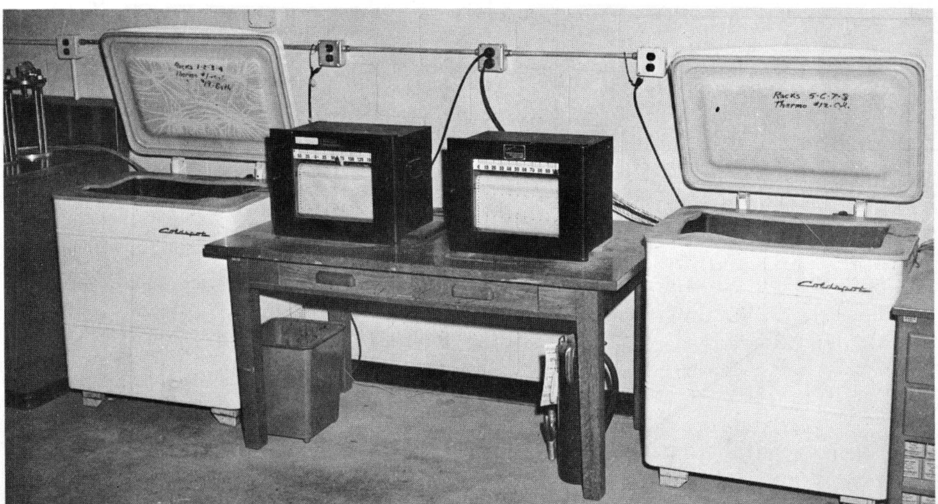

Figure 8. Cooling units and recorders.

in determining simultaneous values of length and temperature. Two typical cooling curves, drawn from recorded data, are shown in Figure 10.

In this report, the expression "dilation" is used to denote an increase in length of test specimens as they are cooled into the freezing range of contained water. Dilation may be transitory; that is, it may not be retained after the specimen has been warmed to the temperature at which cooling was started. Length changes from any cause that are retained when measured at equal temperatures above the freezing point, are referred to as "permanent changes in length." Values of dilation and permanent change in length are reported in terms of millionths (unit change per unit length).

In his discussion, Powers (1) raised the following question and stated that it required an answer based on experimental data before the test procedure can be interpreted properly: In the absence of dilation, is the absorption rate increased significantly by freezing and thawing compared to that obtained by simple soaking? To provide an answer to this question, similar specimens, after drying (drying consisted of storage for 7 days in a closed container over a saturated solution of barium chloride) have been subjected to (a) simple soaking at room temperature, (b) soaking at room temperature with intervening cycles of temperature variation in water in the range of 70 F to 120 F at the rate of 5 cycles per week, and (c) soaking in water at room temperature with intervening cycles of freezing in water-saturated kerosene at the rate of 5 cycles per week. During procedure (c) the temperature was lowered at the rate of 5 F per hour. After the specimens were cooled to 0 F, they were transferred manually to a water bath where they remained at all times except while being frozen. (Test results are shown in Figs. 11 and 12.) Although differences in the amount of water absorbed after drying were not large, schedule (c) produced the greatest dilation and the greatest final permanent change in length. Schedule (c) was adopted as standard for acceptance testing of aggregates.

DURATION OF TEST

The length of the soaking period appro-

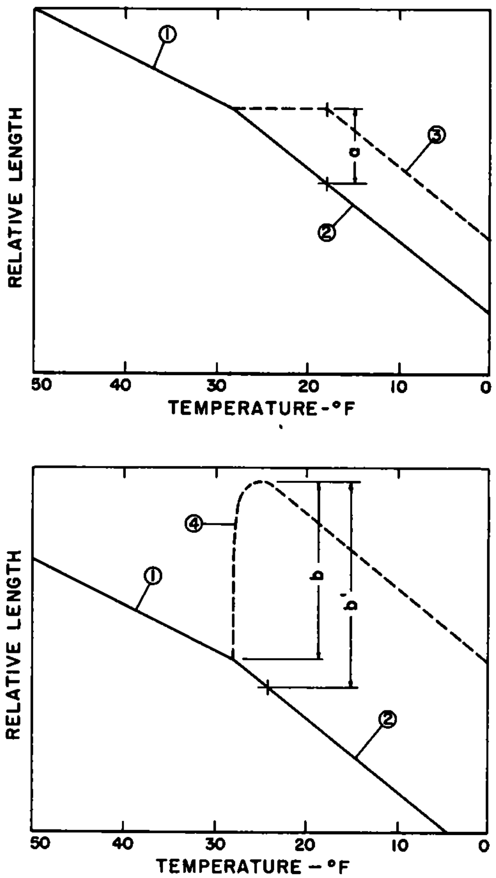

Figure 9. Idealized cooling curves.

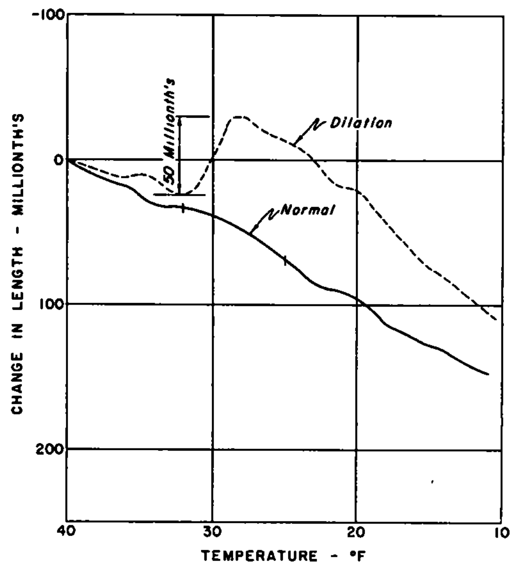

Figure 10. Powers cooling curves illustrating dilation as measured.

priate to the construction site was determined from temperature records of the experimental test slab at Donner Pass. The great majority of freezing-thawing cycles occurred during a 10-week period. For test purposes, therefore, it was concluded that a soaking period of 10 weeks would be appropriate.

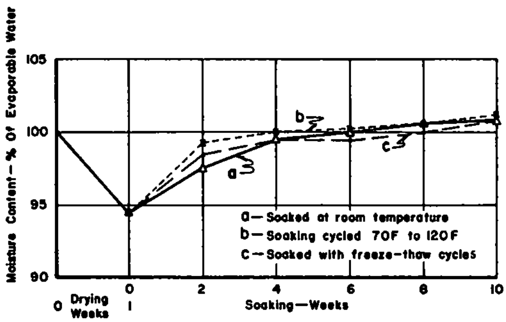

Figure 11. Effect of soaking treatment on absorption (aggregate No. 7).

NUMBER OF CYCLES

Data of the number of freezing and thawing cycles for the winter of 1956-7 are given in Table 2. The freezing point of water in concrete is somewhat less than 32 F. It was concluded that an effective freeze-thaw cycle occurred each time the temperature dropped below 23 F and then rose above 28 F. The data indicate that an estimate of 40 to 50 cycles per year would be amply severe as a criterion for establishing a test procedure. For test purposes, therefore, 40 or 50 cycles of freezing and thawing at the rate of 5 per week were introduced during the 10-week soaking period.

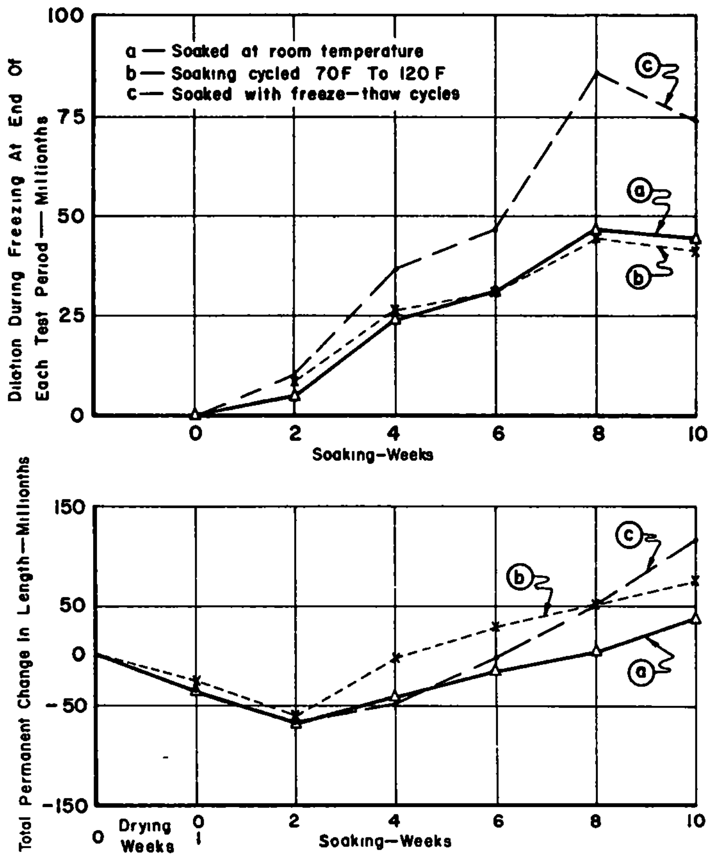

Figure 12. Effect of soaking treatment on dilation and permanent length change (aggregate No. 7).

TABLE 2

SUMMARY OF TEMPERATURE CHANGES WITHIN TEST SLABS[1]

Temperature Range	Depth of Thermocouple Below Surface		
	¼ in	4 in	8 in
(a) Donner Summit, Winter of 1956-7			
Below 28 F Above 33 F	74	41	4
Below 23 F Above 28 F	55	12	5
Below 18 F Above 23 F	30	6	1
(b) Yuba Gap, Winter of 1957-8			
Below 28 F Above 33 F	49	17	7
Below 23 F Above 28 F	23	2	0
Below 18 F Above 23 F	13	0	0

[1]Values shown are number of cycles completed during winter between temperature ranges shown

Except when length changes were being measured during cooling by the Powers procedure, freezing was performed in a larger bath containing water-saturated kerosene, the temperature of which was lowered from about 50 F to 0 F at the rate of 5 F per hour. After the specimens reached 0 F, they were transferred manually to a water bath where they remained at all times except while being frozen. The use of a separate freezing bath for intermittent freezing made it possible to utilize apparatus for the Powers procedure effectively. By properly staggering the program, it was possible to test about 80 specimens concurrently in addition to other specimens receiving simple soaking in water.

END POINT OF TEST

Powers (1) raised the following question which he felt required experimental evidence to answer: Should the end point be the first occurrence of permanent dilation after thawing or the occurrence of dilation (rather than shrinkage) during the freeze, even though the dilation may be small and transitory? The question has been studied by recording length changes during the warming period of the freeze-thaw cycles. Although evidence of hysteresis was noted, it was found that after returning to the starting temperature no permanent increase in length occurred when the measured dilation during freezing did not exceed 50 millionths. It thus appears that dilations of this magnitude or less were the results of stresses within the elastic range, that either no damage occurred to the concrete or that if the damage did occur, it was quickly repaired by autogenous healing.

Later in this report examples will be given of the relationship between dilations in excess of 50 millionths and permanent changes in length.

KEROSENE COOLING BATH

At the start, considerable concern was felt as to the effect of immersing partially dry test specimens in water-saturated kerosene during freezing. A few auxiliary tests indicated the probability that some kerosene was being absorbed. It may be argued that a minor amount of absorbed kerosene would not affect the performance of specimens during freezing since the liquid kerosene should develop hydraulic pressure as does the unfrozen portion of water. Data have been presented to show that regardless of the possible presence of absorbed kerosene, the over-all effect of freeze-thaw cycles during the water soaking period was somewhat more severe than was simple soaking in water.

It is considered to be essential that gain or loss of moisture be prevented during the freezing cycle. Since it is necessary to provide means of contact with the gage studs, the use of a non-aqueous bath is much more convenient than would be a watertight envelope.

In earlier work, specimens at the conclusion of the drying period were immersed at once in kerosene for the measurement of dilation by the Powers procedure. They were then immersed in water except while being frozen in kerosene at the rate of 5 times per week. In later work, the specimens at the conclusion of the drying period were immersed in water at room temperature where they remained for two weeks before they were subjected to freezing and thawing. The probability of absorption of kerosene was thus minimized. This procedure reduced the number of cycles during the 10-week soaking period from 50 to 40. It is not believed that the severity of the test was reduced appreciably by this change.

RAPID WATER TEST RESULTS

During the early stages of the investigation, aggregates from several prospective sources were tested both by the Powers procedure (1) and by rapid freezing and thawing in water, ASTM Designation: C 290-52 T. The aggregates were stream wet when incorporated in the concrete. Specimens were not allowed to dry before subjecting them to the test procedures (Table 3).

It will be noted that aggregates 1 and 2 gave good resistance and aggregates 3 and 7 poor resistance, as measured by both test procedures.

TABLE 3

TESTS OF AGGREGATES BY ASTM DESIGNATION C 290-52 T AND POWERS PROCEDURE

Aggregate No	Average Durability Factor C 290-52 T, 4x5x18-In Prisms	Average Maximum Dilation During Powers Test (10 weeks of soaking) 4½- x 9-In Cylinders
1	90	0
2	77	7
3	21	280
7	43	68

Notes Aggregates were stream wet when incorporated in concrete Specimens were not allowed to dry subsequent to moist curing and prior to start of tests Specimens for both types of test were molded from same batches. Five and one-half-sack concrete, air-entrained Each value is average of 6 or more specimens for the rapid freeze-thaw test and an average of 3 or more specimens for the Powers test

Although the results could be interpreted as indicating adequate resistance to freezing for aggregates 1 and 2, the haul distances to the site of the proposed work were so great as to make their use extremely costly.

In order to explore the effect of drying the specimens subsequent to curing and prior to commencing freezing, another series of rapid water tests was made in which ASTM Designation: C 290-52 T was followed except that test specimens were 4½- by 9-in. cylinders and deterioration was measured by expansion rather than drop in dynamic E. Also part of the specimens were tested without any drying and part were subjected to drying after curing according to schedule E which will be described later. The aggregates tested included numbers 1 and 3 of the first series and several others (Fig. 13).

Data derived from results reported by Kleiger (3) indicate that an expansion of 0.08 percent is approximately equivalent to a reduction of 40 percent in dynamic E. Aggregate 1 exhibited good resistance whether the concrete was given a preliminary drying or not. The remaining aggregates in concrete that were not dried, and therefore tested in this respect in accordance with ASTM Designation C 290-52 T, failed rapidly. Also, a moderate amount of drying produced a marked improvement in resistance.

Individual specimens subjected to the ASTM rapid water test were quite erratic in performance. The reliability of average results (Table 3 and Fig. 13) is therefore subject to question. Uncertainties in interpretation of the ASTM rapid water test led to its discontinuance. Subsequent studies were devoted to the development of Powers procedure (1).

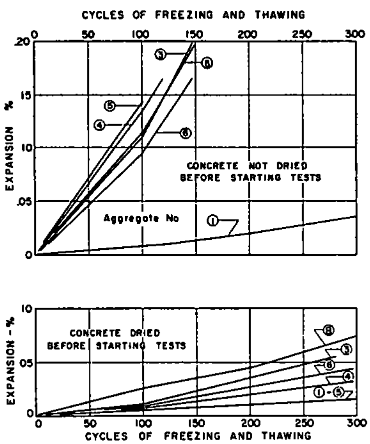

Figure 13. Rapid freezing and thawing in water. Similar to ASTM Designation: C 290-52T except using 4½- x 9-in. cylinders which were measured for change in length.

CORRELATION WITH FIELD PERFORMANCE

A measured dilation in excess of 50 millionths (in. per in.) above the length at the apparent freezing point of water in the specimen has been adopted as the criterion of unsatisfactory dilation. Concrete that produces less dilation during the selected time of the test is reported to be satisfactory. The validity of the selected criterion can be confirmed or rejected by the performance in service of concrete so tested.

To date, 1,242,000 sq yd or 180 lane-miles of pavement have been constructed under six contracts on the Donner Pass route between Colfax and the Nevada state line. Of this amount, 173 lane-miles have been subjected to one or two winter's exposure. Salts have been applied during the winter to maintain the pavement in a substantially ice free condition. Aggregates used in this work were accepted on the basis of the Powers procedure as described earlier (details of the latest specification requirements are given in the appendix). The aggregates used in three of the projects, (aggregates 4, 7 and 8) were tested by rapid freezing and thawing in water, ASTM Designation: C 290-52 T and, (Table 3 aggregate 7 and Fig. 13 aggregates 4 and 8) would not be considered by this test to be resistant under any reasonable interpretation of the data.

The authors have examined all of these pavements in detail on several occasions. It is their conclusion that there is no significant evidence of distress attributable to the action of freezing and thawing. This statement warrants further amplification because of the possibility that others who might have occasion to inspect the work might consider that certain defects are indications of distress due to freezing and thawing. In a project between Boca and Floriston, (a few miles west of the Nevada state line) rather severe raveling occurred at several locations during the winter of 1959-1960; the first winter after construction. In other locations on this project, surface mortar has become detached to a depth of about $\frac{1}{16}$ in. Most of the distress is in the outer lanes. It has been observed that raveling starts abruptly at locations such as bridge approaches or at the start of a days work. The pattern is such as to suggest that local differences in performance are due to variation in construction methods, possibly erratic curing, rather than to the quality of the materials that were used. The authors have concluded that surface abrasion, where it has occurred, is the result of mechanical action of tire chains.

A section between Hampshire Rocks (elevation 5,800 ft) and Soda Springs (elevation 6,800 ft) west of Donner Pass constructed in 1959, is of particular interest. This pavement is believed to be in the most severe location of the route. It traverses a mountain meadow with heavy vegetation giving evidence of abundant precipitation. Freezing and thawing cycles are substantially the same in number as those at the summit. Aggregate No. 7 was used in the concrete. This aggregate, although meeting the selected criterion by the Powers test, performed poorly in the ASTM rapid water test. The pavement has been examined carefully at many closely spaced locations. Three types of defects have been noted. One is the presence of pieces of wood in the surface of the pavement. The second is a pit 1 in. or larger in diameter which was caused by a mechanically weak particle of rhyolite tuff. The pit edges are sharply defined by the surrounding concrete. These pits appear at the rate of 0 to 5 per 12- by 15-ft slab. The third defect is a typical popout produced by an unsound particle a short distance below the surface. Rupture of the overlying concrete has produced a crater-like depression. This type is absent in many areas and when present, has not appeared with a frequency greater than one per 12- by 15-ft slab.

Except as described above, the authors have observed nothing in any of the pavements that indicates freeze-thaw distress due to the materials used.

VARIATIONS IN DRYING

The concept of preliminary drying of cured test specimens before subjecting them to cycles of freezing and thawing is based on the expectation that pavements would be constructed during the summer or early fall and would have some opportunity to dry before the onset of severe weather. If construction were to be completed so late in the fall that a lesser degree of drying occurred before severe weather, the adopted schedule of drying the test specimens would produce unrealistic results.

The effect of varying degrees of drying of test specimens has been explored with aggregate No. 7. Five degrees of drying after standard curing were investigated.

After moist curing for 14 days, the specimens were conditioned by placing them in sealed containers over saturated solutions of salts which produced atmospheres of the relative humidities given in the following five schedules of drying:

A. No drying (cured 3 weeks under standard fog conditions).

B. Seven days over sodium sulfate, 97 percent relative humidity.
C. Seven days over barium chloride, 87 percent relative humidity.
D. Seven days over sodium acetate, 70-75 percent relative humidity.
E. Two days in air at 50 percent relative humidity and 5 days over sodium acetate.

Schedule E was used in acceptance testing for the construction work completed to date. The different schedules resulted in varying losses in water during drying in grams per specimen as follows:

A. + 3 grams (gain),
B. -15 grams,
C. -18 grams,
D. -53 grams, and
E. -87 grams.

As shown in Figure 14, schedules A, B and C resulted in dilation of 50 millionths or greater after ½, 3½ and 6 weeks of soaking, respectively. Schedules D and E did not result in dilation as great as 50 millionths during the 10-week soaking period.

Total permanent changes in length are shown in Figure 15. Increases in length above that at the conclusion of moist curing resulted from schedules A, B and C in decreasing order of magnitude at the end of 10 weeks of soaking. Schedule D resulted in a length equal to that at the conclusion of moist curing. The length produced by schedule E, while greater than at the conclusion of the drying period, was less than the "as-cured" length.

The results show that relatively small variations in drying procedure cause greatly different degrees of resistance to freezing and thawing. As a result of this investigation schedule D is now being used for acceptance purposes for new construction in the Donner Pass area in lieu of schedule E which has been used formerly.

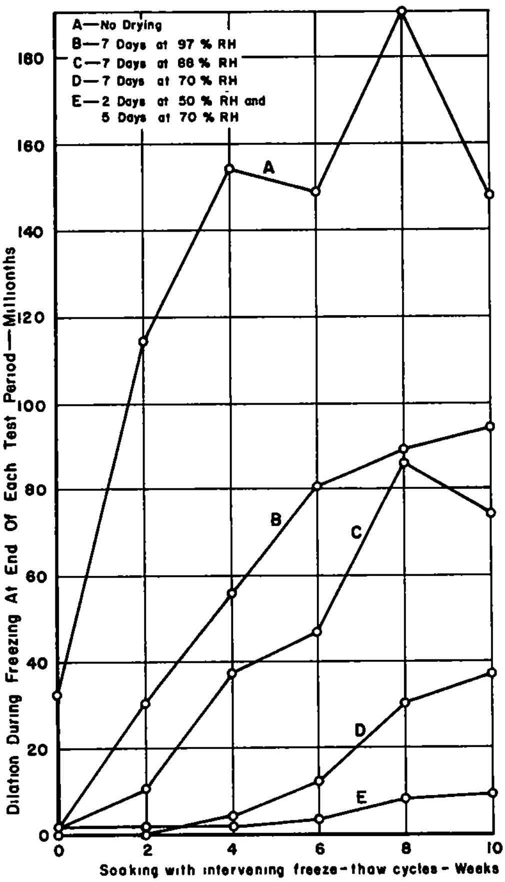

Figure 14. Effect of partial drying on dilation (aggregate No. 7).

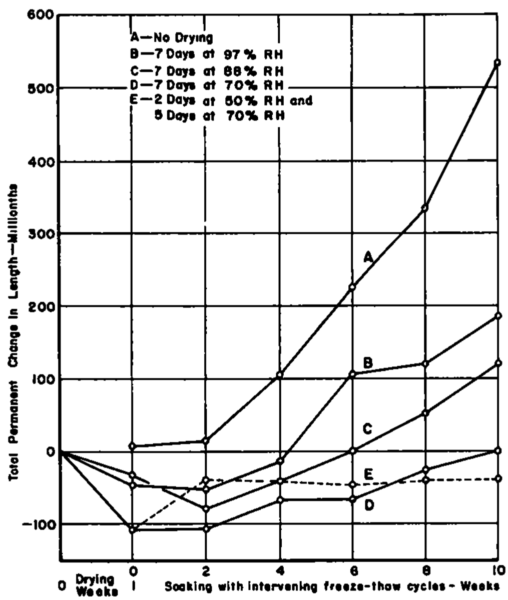

Figure 15. Effect of partial drying on permanent change in length (aggregate No. 7).

IOWA LIMESTONE

The test performance of limestone from the Rapid formation in Iowa was explored in concrete in which the fine aggregate consisted of sand from the American River (aggregate No. 1). The limestone as received was a crushed product in a dry condition. It was sieved and recombined to 1½ in. to No. 4 sieve. The limestone was prepared for incorporation into concrete by two methods: (1) soaking in water at room temperature for 24 hr, and (2) saturation under vacuum.

After the specimens were fog cured, they were subjected to conditioning schedules A (no drying) and E (drying).

Figure 16 shows the observed dilations during the Powers cooling cycle. Vacuum saturation produced more rapid distress than did simple soaking but either treatment resulted in definite indications of poor durability even when the concrete was subjected to preliminary drying.

Figure 17 shows permanent changes in length of the specimens. The curves again show a rapid loss in durability as the concrete was soaked.

With the limestone, when incorporated into concrete after 24 hr of soaking, specimens that were not dried performed nearly as well as those that had been partially dried after curing. This is in direct contrast with results obtained under similar treatments with aggregate No. 7 which was incorporated in the concrete in a stream-wet condition. It appears that the limestone was not completely saturated by 24 hr of soaking, and therefore, was in a condition approaching that resulting from drying the concrete after curing. It also appears that the limestone slowly absorbed water from the paste as evidenced by continued shrinkage for several days after the specimens were immersed in water, as shown in Figure 17 (concrete not dried).

LONG-TIME SOAKING

On the completion of tests by the Powers procedure (1) which were discontinued at the end of the 10-week soaking period, the specimens were placed in water storage at room temperature where they remained for periods up to three years. A cooling curve was again recorded for some of these specimens. (Tests results are shown in Figure 18.) Specimens that had received no preliminary drying were rendered more vulnerable. This can be explained by the

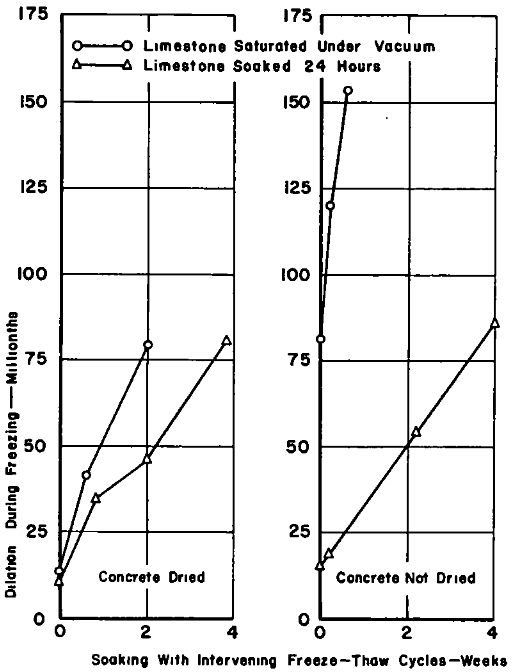

Figure 16. Effect of aggregate condition and drying of concrete on dilation— Iowa limestone coarse aggregate.

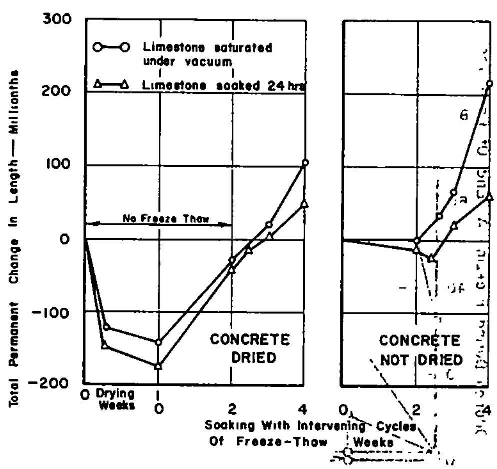

Figure 17. Effect of aggregate condition and drying of concrete on permanent changes in length— Iowa limestone coarse aggregate.

theory that the cement during hydration extracted some of the water from the presumably completely saturated aggregate. During prolonged soaking, part or all of this water was restored. Specimens that had been subjected to preliminary drying were also rendered more vulnerable but at a lower rate. None of the dried specimens reached a critical dilation of 50 millionths during the period of soaking, but the trend of results points to the conclusion that concrete is not likely to remain permanently resistant to freezing if it is exposed in such a way that it is continuously immersed in water.

LONG-TIME SOAKING FOLLOWED BY A SHORT DRYING PERIOD

A few specimens containing aggregate No. 7 were soaked in water for 9 mo following completion of tests by the Powers procedure. Cooling curves then showed dilations of 80 and 110 millionths for specimens dried originally in atmospheres of 70 and 87 percent relative humidity, respectively. The specimens were again dried for 8 days in an atmosphere of 70 to 75 percent relative humidity. Cooling curves then showed that dilations were reduced to 43 and 49 millionths. These results show that even short periods of exposure to mild drying conditions provide substantial relief to the build-up of vulnerability resulting from long-time soaking.

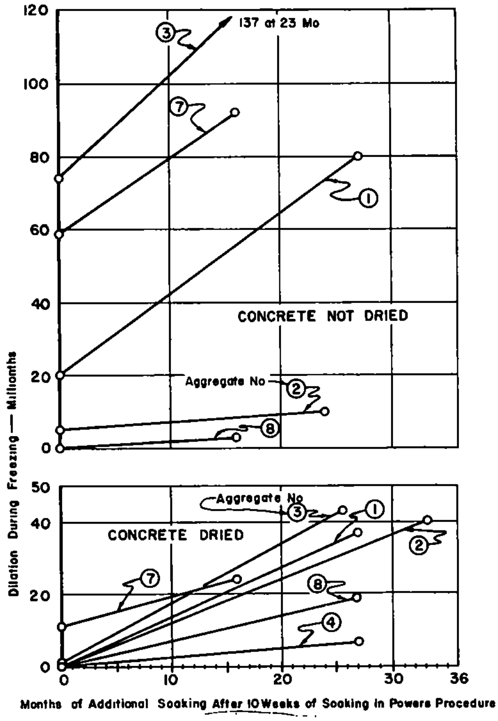

Figure 18. Effect of long soaking on dilation.

EVALUATION BY PERMANENT LENGTH CHANGE

Examples of permanent changes in length of specimens are shown in Figures 12, 15 and 17. The feasibility of using such length changes as criteria of the performance of aggregates in concrete has been studied. Concrete shrinks when drying and swells when soaking. During all cycles of simple soaking, swelling is less than the original shrinkage. When subjected to freezing and thawing cycles in conjunction with soaking at room temperature, the amount of swelling may be greater than under simple soaking. An increase above normal swelling may indicate damage as a result of freezing.

In many cases, those specimens that developed a dilation in excess of 50 millionths during cooling have attained a permanent length approximately equal to the as-cured length. If this were a universal rule, it would be possible to eliminate the test for dilation during cooling and to use only the measurement of permanent length change as a criterion of performance. To do so, would permit the elimination of expensive and complicated apparatus for recording the cooling curve. However, some specimens have not swelled to the as-cured length at the time a critical dilation has been found in the cooling curve. In other cases, the length of the specimen has exceeded the as-cured length before a dilation of 50 millionths has been indicated by the cooling curve. Attempts have been made to relate permanent changes in length as referred to the as-dried length and dilations as indicated by the cooling curve. The results were negative.

It appears therefore, that evaluation of performance in accordance with the Powers concept must include the measurement of dilation during cooling. Manual, in place of automatic measurement, could be used with considerable saving in the cost of apparatus.

EFFECT OF CEMENT FACTOR

The movement of water from and into aggregate particles is restricted by the surrounding cement paste. The more impervious the paste, the greater should be the restriction on movement of water into and out of the aggregate. Richer mixtures (of lower water-cement ratios) therefore, should extend the period during which partially dried concrete can be soaked before it becomes vulnerable to the effects of freezing and thawing.

If the above assumptions are true, higher cement factors should produce concrete that is more resistant to freezing and thawing under natural exposure. Reported examples of field performance as affected by cement factor are few in number; however, certain projects of the "Long Time Study of Cement Performance of Concrete" (4) have yielded such comparisons. In the "Ten Year Report" (4), it is stated that one row of boxes at the Illinois test plot containing 27 test cements and an aggregate of good service record, had developed considerable distress. These boxes were constructed with non-air-entrained concrete containing 4½ sacks of cement per cubic yard and having a slump of 8 in. Boxes constructed of similar concrete except with a cement factor of 6 sacks per cubic yard, were in excellent condition. At the Saugerties, New York test site (5) concrete piles of 7-sack concrete have been more resistant to freezing and thawing than comparable piles containing 5-sack concrete.

A good laboratory test procedure should be capable of exhibiting improved performance of richer (lower water-cement ratio) concrete. The performance of the Powers procedure in this respect has been investigated. Specimens were made of concrete containing 4, 5½ and 7 sacks of cement per cubic yard using aggregate No. 7. The corresponding water-cement ratios were 7.5, 5.1 and 4.3 gal per sack, respectively.

After moist curing for 14 days, groups of 6 specimens were subjected to four schedules of drying; namely, schedules A and E as previously described and two intermediate conditions which were similar to, but not exactly the same as, schedules B and C. The intermediate drying conditions are designated as schedules B' and C'. After curing and drying according to these schedules, the specimens were subjected to 14 weeks of soaking, of which the first two weeks were in water at room temperature and the remainder were with intervening cycles of freezing and thawing at the rate of 5 per week.

The results of the tests are summarized in Tables 4 and 5.

Referring first to the data on change in weight (Table 4) which is used as a measure of moisture movement, the results are reported to the end of 8 weeks of soaking only, because at later periods, the mechanical loss of solid material became great enough to obscure the moisture change relationship. It will be noted that the loss of moisture during drying increased with decreasing

TABLE 4

EFFECT OF CEMENT FACTOR ON MOISTURE MOVEMENT (Aggregate No. 7)

Cement Factor (sk/cu yd)	Drying Schedule A[1]	B'	C'	E
	(a) Change in Weight (grams per specimen) Due to Drying			
4	4	-38	-66	-88
5.5	3	-29	-49	-67
7	3	-24	-37	-52
	(b) Change in Weight (grams per specimen) Relative to the As-Dried Weight After 8 Weeks of Soaking			
4	-	40	58	77
5 5	-	43	52	67
7	-	37	41	54
	(c) Change in Weight (grams per specimen) Relative to the As-Cured Weight After 8 Weeks of Soaking			
4	20	2	- 8	-11
5 5	18	14	3	0
7	15	13	4	2

[1]Schedule A specimens were soaked in water for one week while remaining specimens were subjected to drying.

TABLE 5

EFFECT OF CEMENT FACTOR ON DILATION AND PERMANENT CHANGE IN LENGTH (Aggregate No 7)

Cement Factor (sk/cu yd)	Drying Schedule A	B'	C'	E
	(a) Time of Soaking (weeks) to Produce 50 Millionths Dilation			
4	5	5	8	7
5 5	3	4	11	12
7	2	9	12	14+
	(b) Total Permanent Change in Length (millionths) Relative to As-Cured Length After 14 Weeks of Soaking			
4	160	105	40	15
5 5	240	65	-45	10
7	230	-40	-25	-55

cement factor, a result that is in accordance with theory. Upon soaking, the leaner concrete absorbed more water. At the end of 8 weeks however, the leaner concrete that had been subjected to drying schedules B', C' and E contained less water than the richer concrete.

Judged on the basis of final moisture content, it might be concluded that the leaner concrete, after drying and then soaking, should be more resistant to the effects of freezing and thawing. That this is not true, however, is shown by the data of cooling curves (Table 5). These data show that with increasing cement factor, the concrete withstood increasingly longer periods of soaking before the specimens became sufficiently vulnerable to produce a dilation of 50 millionths. A reversal of this trend is shown for specimens that were not dried (schedule A). The reversal in trend may be explained by the fact that since the aggregate as incorporated in the concrete was thoroughly saturated, further soaking did not affect its vulnerability greatly. All cement factors under this condition produced highly vulnerable concrete and the slight differences in safe periods of soaking are of little practical consequence.

The data of permanent change in length (Table 5) show that after drying, the richer concrete was more resistant. For specimens that were not dried (schedule A), the data indicate somewhat better performance for the 4-sack concrete; however, its resistance was of low order and the differences between cement factors are of little practical consequence.

On the whole, the data give convincing evidence that the test procedure exhibited improved performance with increasing cement factor, and decreasing water-cement ratio. The fact that this behavior is in accordance with observed performance under natural exposure lends added evidence of the validity of the basic concepts of the Powers method.

REPEATABILITY

The degree to which test results can be duplicated in the same laboratory is a matter of considerable importance in assessing the value of any test method.

The aggregates that have been tested, with one exception, are sands and gravels of igneous origin and are composed of intrusive and extrusive acidic, intermediate and basic rocks. Some of the particles have absorptions as high as 5 percent. The problem of making like specimens is therefore, connected intimately with that of getting representative amounts of various rock types into them. When large dilations have occurred they have been accompanied frequently by rupture of the specimen apparently caused by abrupt expansion of one or more large particles (Fig. 19). The fact that the larger particles are the more vulnerable further complicates the fabrication of uniform test specimens.

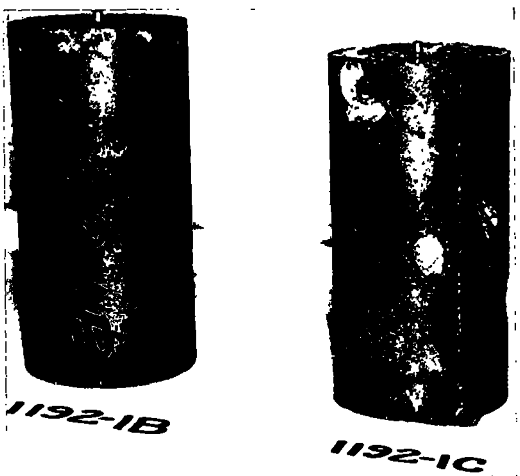

Figure 19. Effect of drying concrete on condition of specimens subjected to freeze-thaw (1192-1B dried after moist curing, and 1192-1C not dried).

For this reason, specifications have been written on a go-no-go basis. That is, acceptance has been based on the condition that the majority of the individual specimens pass the test requirement. A number of aggregates have been accepted as a result of tests of six specimens, two each from three batches mixed on different days. Usually more than one sample from the same deposit has been tested.

Six is certainly the minimum number of specimens that should comprise one test. Twelve or more is desirable. The reason for using a number as small as six in past work has been to provide extra specimens for study purposes on the effect of variations in drying or soaking procedures. The volume of the 4½- by 9-in. cylinder that has been used is about 0.083 cu ft. A test batch of 0.6 cu ft is ample to determine slump, air content and yield, preparatory to making four tests specimens. Twelve specimens can, therefore, be obtained from three batches of this size.

The measure of repeatability should be based on (1) the variation in time of soaking required to exceed a selected degree of dilation, or (2) the percentage of specimens that reach or do not reach the critical dilation in the specified period of soaking.

A measure of the degree of variation between individual specimens under category (1) is obtained by comparing the soaking time of individual cylinders to reach specified amounts of dilation measured during the Powers procedure. Since the cooling curve has been recorded at two-week intervals, it is possible that dilations during intervening freezings may exceed those at the time of measurement. Tests of individual specimens have shown instances of large dilations being followed by smaller ones. Nevertheless, by plotting the data obtained, the time of soaking can be fairly well estimated to about ¼ week for each increment of dilation considered. A few sets of test data were selected from those groups in which all of the test cylinders dilated more than 50 millionths during the 10-week soaking period, and the weeks of soaking to reach dilations of 30, 40 and 50 millionths were determined. The standard deviation for each group was calculated also, and the results are given in Table 6.

The above method of evaluating precision is severe because the aggregates tested were composed of a variety of rock types, some of which could produce a small and transitory dilation at the time of measurement largely as a matter of chance inclusion in different specimens. As the dilation selected for consideration becomes larger, and more significant, the variation in time between specimens becomes smaller percentage-wise.

A more useful measure of repeatability is obtained by considering the results on a go-no-go basis as suggested under category (2).

As mentioned earlier, some sources were tested more than once at different times. Aggregate No. 7 was tested early in 1958 and again late in 1958. Two test pits were sampled for the early series, and three for the latter, bringing the total number of samples tested to five. The tests for development of dilation were made under the same conditions for each sample. The results of the test are given in Table 7 which shows the number of samples from each group that pass the test. Six specimens were used in each test in which the concrete was dried. With the exception of the second sample of the not-dried groups, five specimens were used. Only four specimens were used in the second sample. The results are amply discriminatory on the basis of evaluation on the performance of at least two-thirds of the specimens.

That the test method, even with the relatively small number of test specimens, effectively distinguishes between vulnerable aggregate and nonvulnerable aggregate for the test conditions used, is given in Table 8. The sources listed are those tested recently under the current specification requirements, and also without drying. In all cases involving drying of the concrete, at least 5 of the 6 specimens definitely passed the test. Of the not-dried group, the separation was positive except for source 11 in which 1 of 5 passed and sources 14 and 15

TABLE 6

VARIANCE IN RATE OF DILATION OF SELECTED TEST GROUPS

Test Group	Time to Reach Specified Dilation (weeks) and Standard Deviation for Each Group					
	30 Millionths		40 Millionths		50 Millionths	
	Wks	Std Dev	Wks	Std Dev	Wks	Std Dev
A	1.6	1.6	2.6	1.5	3.6	1.5
B	0.6	0.6	1.3	0.6	1.6	0.5
C	0.6	0.2	1.0	0.3	1.4	0.4
D	3.6	2.3	6.5	1.6	7.2	1.8

TABLE 7

GO-NO-GO EVALUATION OF FIVE AGGREGATE SAMPLES FROM SAME SOURCE (Aggregate No 7)

Sample No	Condition of Concrete	
	Dried, Then 10 Wks of Soaking	Not Dried, 11 Wks of Soaking
1	6 of 6 pass	0 of 5 pass
2	5 of 6 pass	1 of 4 pass
3	6 of 6 pass	0 of 5 pass
4	6 of 6 pass	0 of 5 pass
5	6 of 6 pass	0 of 5 pass

in which 3 of 5 passed in each case. It is evident that better repeatability was obtained with dried specimens than with undried ones.

The foregoing discussion indicates that the test procedure, based on the Powers concept, is sufficiently precise to render it usable as a basis for purchase specifications for concrete aggregates.

TABLE 8

GO-NO-GO EVALUATION OF SEVERAL AGGREGATES

	Condition of Concrete	
Aggregate No	Dried, Then 10 Wks of Soaking	Not Dried, 11 Wks of Soaking
9	6 of 6 pass	5 of 5 pass
10	6 of 6 pass	5 of 5 pass
11	5 of 6 pass	1 of 5 pass
12	5 of 6 pass	0 of 5 pass
13	6 of 6 pass	0 of 4 pass
13A[1]	5 of 6 pass	0 of 5 pass
14	6 of 6 pass	3 of 5 pass
15	6 of 6 pass	3 of 5 pass

[1]Aggregate No 13A contains coarse aggregate No 13 and fine aggregate No 15

CONCLUSIONS

The method of performing freezing and thawing tests to determine performance of aggregates in air-entrained concrete as suggested by Powers (1), is adaptable to different methods of producing and handling aggregates and to varying exposure conditions. Results in service show that when the test conditions were adjusted to the aggregates as they were furnished and to the exposure conditions to which pavements were subjected, aggregates that did not produce a dilation in excess of 50 millionths have performed satisfactorily through one and two winters of severe weather. There is no present evidence to warrant an expectation that the concrete will not continue to be durable.

On the other hand, had acceptance of the aggregates been based on tests by ASTM Designation: C 390-57 T, or Corps of Engineers Methods CRD-C 20-55, Resistance of Concrete Specimens to Rapid Freezing and Thawing in Water, and had the concrete been mixed with aggregates containing the full amount of original moisture, several of the aggregates that were used would have been rejected. Rejection of such aggregates would have increased the cost of the work substantially.

REFERENCES

1. Powers, T.C., "Basic Considerations Pertaining to Freezing-Thawing Tests." Proc., ASTM, 55:1121 (1955).
2. Powers, T.C., "The Physical Structure and Engineering Properties of Concrete." Research and Development Laboratories, Portland Cement Association, Chicago, Ill., Bul. 90.
3. Klieger, Paul, "Effect of Entrained Air in Strength and Durability of Concrete Made with Various Maximum Sizes of Aggregate." Proc., HRB, 31:177-201, Table 15 (1952).
4. "Ten-Year Report on the Long-Time Study of Cement Performance in Concrete." Advisory Committee, Long-Time Study of Cement Performance in Concrete, R.F. Blanks, Chairman, Proc., ACI, 49:601 (1953).
5. Tyler, I.L., "Long-Time Study of Cement Performance in Concrete—Chapter 12—Concrete Exposed to Sea Water and Fresh Water." Proc., ACI, 56:825 (1960).

Appendix

METHOD OF TEST FOR FREEZE-THAW RESISTANCE OF AGGREGATES IN AIR-ENTRAINED CONCRETE (Powers Procedure)

Values shown for maximum size of aggregate, size of test specimen, conditioning by drying and length of soaking period are those used by California Division of Highways for construction work in a particular locality. Other values should be substituted when construction or exposure conditions so warrant.

Samples

Samples of aggregates shall be secured under the direct supervision of the engineer in charge of the work. Samples from existing stockpiles of processed aggregate shall be taken from washed materials and shall be visibly damp. Samples from materials in place in a proposed source shall be taken at depths from the surface that will insure the presence of the full quantity of ground water. Excavations for the purpose of securing samples shall be made to the full depth of intended operations. Samples shall be protected against loss of contained water until they are delivered to the engineer.

Samples shall be shipped to the laboratory in metal containers with tight fitting covers. Water shall be added to each container before it is sealed.

In the laboratory, samples shall be so handled to prevent loss of absorbed water. They shall be processed by washing if required, separated into sieve sizes and recombined to the required grading in the presence of excess water. If particles larger than 1½ in. are present in the sample, they shall be crushed and added to the finer material unless it is proposed to waste the oversize particles during manufacture.

Apparatus

The following apparatus is required in addition to that needed for making and curing concrete specimens in the laboratory, ASTM Designation: C192.

One or more refrigerated baths of a size and depth required to contain the test specimens immersed in kerosene and with suitable controls to provide a lowering of the temperature at a rate of 5 ± 1 F per hour from room temperature to 0 F. Household type deep freezing chests with a copper liner have been found to be satisfactory.

One or more water baths of a size and depth to contain the specimens immersed in water, equipped with water supply and overflow outlet.

A supply of frames with linear variable differential transformers attached, for supporting test specimens for automatic measurement of length changes during cooling. A satisfactory design is shown in Figures 5 and 6.

A supply of 4½- by 9-in. cylinder molds, equipped with top and bottom detachable plates for holding gage studs.

A comparator for measuring permanent length changes of test specimens, meeting the requirements of Section 2(b) of ASTM Designation: C 157-54 T and with a standard reference bar.

A balance sensitive to 1 g and having a capacity of about 6,000 g.

Closed corrosion resistant containers large enough to contain one or more 4½- by 9-in. test specimens with free space of at least ¾ in. from all faces, equipped with means of supporting the specimens at least 2¼ in. above the bottom.

A strain recorder with suitably ruled paper. It shall consist of a multipoint displacement recorder having one channel for each specimen being tested for dilation during cooling. The switching sequence from one channel to the next shall be accomplished automatically at a rate such that the time interval between prints for a particular channel does not exceed 5 min. The system shall have a sensitivity sufficient to indicate displacements of 25 millionths inch equal to one chart division through a range of 0.004 in. Displacements indicated by the linear variable differential transformers in contact with the studs of test specimens shall be recorded. A calibration bar of accurately known thermal coefficient of expansion shall be furnished. (Manual means of measuring length changes and temperature may be substituted for automatic recorders provided the apparatus has a sensitivity equal to that of the automatic apparatus.)

An automatic temperature recorder connected to thermocouples imbedded in companion concrete cylinders of the same size as the test specimens, one in each bath in which dilations are measured. The temperature of each thermocouple shall be printed on a chart within an accuracy of 1 F through a range from room temperature to -10 F. Prints for each channel shall be recorded at a frequency of not more than 5 min.

Procedure

Concrete mixtures shall be proportioned with 1½-in. maximum size aggregate graded to conform to the specifications for the work. The cement factor and slump shall be within the limits specified for the work. An air-entraining agent consisting of neutralized

Vinsol resin solution shall be incorporated in the quantity required to result in an air content of 4.5 ± 0.5 percent air in the fresh concrete. Slump, air content and cement factor shall be determined for each batch. The size of batch shall be sufficient to provide at least four 4½- by 9-in. test specimens.

Four or more 4½- by 9-in. cylinders shall be molded from each batch. Stainless steel gage studs 1 in. long shall be embedded in the fresh concrete so as to project ⅛ in. at each end of the longitudinal axis of the specimen.

For each aggregate or combination of aggregates to be tested, at least three batches of concrete shall be mixed, each on a different day, providing a minimum of 12 test specimens. The specimens in the molds shall be cured under standard moist conditions for 24 ± 4 hr and then be removed from the molds. Standard moist curing shall be continued to the age of 14 days.

At the end of the moist curing period, the specimens shall be weighed to the nearest 1 g in a surface-dry condition and shall be measured for length to the nearest 0.0001 in. (The method of inserting gage studs does not insure that they coincide absolutely with the vertical axis of the specimen. For this reason the needle of the dial gage may not remain stationary as the specimen is rotated. Reproducible results are obtained by rotating the specimen until the minimum reading is found.) They shall then be placed in closed containers over a saturated solution of sodium acetate with an excess of salt for 7 days at a temperature of 73.4 ± 3 F. The solution shall be at least 1½ in. in depth and shall not be closer than ¾ in. to the bottom of the test specimens. The specimens shall be removed, weighed and measured for length. They shall then be immersed in water at room temperature for 14 days.

The specimens shall then be removed from the water bath (total age at this point is 5 weeks). They shall be wiped free of surface water and be weighed and measured for length.

The specimens shall then be placed in the frames used to measure and record strains during cooling. The specimens in the frames shall be immersed in water-saturated kerosene in the refrigerated baths and connected to the displacement recording instrument which shall be placed in operation. A companion specimen containing a thermocouple shall also be placed in the bath and be connected to the temperature recording instrument which shall be placed in operation. The kerosene shall be agitated mechanically until the temperatures of the specimen and the bath are equal. The temperature at this point shall be above 45 F. The time shall be recorded on the charts. The bath shall be cooled at a rate of 5 ± 1 F per hour and cooling shall be continued until the temperature reaches 0 ± 5 F. The specimens shall then be removed and the time recorded on the charts.

The specimens shall be immersed in water and allowed to warm to room temperature, then weighed and measured for length. The specimens shall remain in the water bath continuously except that they shall be cooled in kerosene as described above, at the rate of five times per week until a total soaking period of 10 weeks has elapsed. (The total age of the specimens at this point is 13 weeks.) Cooling to 0 F except during the first cycle and each succeeding 10th cycle may be performed without connecting the specimens to the strain-recording device. At the end of each two-week soaking period, the specimens shall be weighed and measured for length and then be connected to the strain-recording device during one cycle of cooling.

At the conclusion of each recorded cooling cycle, the charts shall be marked to show equal time periods of 15 min. The recorded strain, estimated to the nearest one-half chart division, divided by the gage length, 7½ in., shall be plotted against the temperature recorded at the same time. The curve shall be examined for evidence of dilation at the approximate freezing point. If dilation is evident, it shall be measured as the distance between the start of dilation at the apparent freezing point and the greatest succeeding length. The result shall be recorded as the dilation in millionths.

Report

Specimens shall be reported to have "passed" the test if:

1. The dilation at any measured period did not exceed 50 millionths.

2. If the permanent length at the conclusion of the soaking period does not exceed the length at the conclusion of the 14-day moist curing period by more than 0.006 percent of the gage length (measured between inner ends of the gage studs).

Specimens shall be reported to have failed the test if they fail to meet any of the requirements set forth above to qualify as "passing".

The aggregate or combination of aggregates under test shall be reported to have "passed" the test if 65 percent or more of the individual specimens of the group passed the test. If less than 65 percent of the individual specimens of the group passed the test, the aggregate or combination of aggregates shall be reported to have "failed" in the test.

A Study of Chert and Shale Gravel in Concrete

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Certain chert and shale gravels have long been recognized as harmful when included in portland cement concrete exposed to freezing and thawing. Many organizations have specifications limiting percentages of these materials in concrete aggregates, but few of these specifications distinguish between types of chert and shale from different geographical areas nor do they always take into account the basic physical properties of these materials.

In this study, pore characteristics, mineralogy, texture, and structure were determined for cherts and shales from nine Indiana glacial gravel deposits by means of microscopic petrography, X-ray diffraction, and the common specific gravity and absorption techniques. Blends of 2, 4, 6, and 10 percent of chert or shale from each source were made with a standard durable crushed limestone coarse aggregate, and these blends were used in 3- x 4- x 16-in. air-entrained concrete beams subjected to up to 300 cycles of freezing and thawing. A measure of the amount of deep-seated deterioration of the beams was provided by durability factors calculated from the results of non-destructive sonic testing of the beams at intervals during freeze-thaw testing. Severity of surface deterioration was also evaluated. The influence of the basic properties of the chert and shale gravels on the results of the freeze-thaw tests was then determined. On the basis of the results of these tests, the existing specifications on cherts and shales were studied to determine whether the specifications realistically categorize these materials.

Despite significant differences in their mineralogies, no difference was noted in the freeze-thaw durabilities of the various chert samples. For all the cherts, significant deep-seated and surface deterioration occurred only in beams containing 6 to 10 percent of material with a bulk specific gravity (saturated surface-dry basis) of less than 2.45.

Although the basic properties of the shales varied even more widely than those of the cherts, none of the shales caused deep-seated failure of the concrete. However, the most porous shales caused "popout" damage, which was especially severe at the 6 and 10 percent levels.

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THERE IS still much to be learned about the physical properties of cherts and shales, and their effects on the durability of concrete in spite of considerable past study.

In the past little has been done to differentiate between cherts and shales of the same general type, but that are obtained from different geographic areas. A purpose of this study was to determine if the basic properties of cherts and shales from one part of Indiana differ significantly from those of cherts and shales from other parts of the state, and if significant differences in the properties of these materials were found, to attempt to determine if these differences also result in differences in durability.

Another objective was to quantify the effects of different chert and shale gravels on the freeze-thaw durability of concrete test specimens containing small percentages of these materials.

The cherts and shales used in this investigation were obtained by hand picking from glacial gravel deposits widely scattered throughout the State of Indiana. In this way six chert and five shale samples, constituting as widely divergent a group of cherts and shales as could be found in Indiana, were obtained. The geographic locations of the sources for these samples are shown in Figure 1.

In order to determine the effect of presence of the deleterious materials on freeze-thaw durability of concrete, small percentages of the cherts and shales being studied were incorporated in 3- by 4- by 16-in. air-entrained portland cement concrete beams made with a standard portland cement, crushed stone coarse aggregate, and natural sand fine aggregate. The aggregates were obtained from sources of proven good quality. The only variables purposely introduced into the experiment were the deleterious materials themselves. The mix design was held constant for all beams made except that varying small percentages of chert or shale were substituted for part of the crushed limestone coarse aggregate in all but the control beams. A water-cement ratio of 0.46 by weight was used throughout the study. This water-cement ratio produced a mix with good workability and a slump of about 3 in. The cement factor was kept constant at six bags per cubic yard. An air-entraining agent was used to entrain approximately 4 percent air in each batch.

In all beams the coarse aggregate was used in equal amounts of the No. 4 to 3/8-, 3/8- to 1/2-, 1/2- to 3/4-, and 3/4- to 1-in. sizes. Chert or shale was substituted for the 3/8- to 1/2-, 1/2- to 3/4-, and 3/4- to 1-in. crushed stone in 2, 4, 6, and 10 percent blends in all but a few control beams in which the coarse aggregate consisted of 100 percent crushed limestone. In the beams containing the deleterious materials, no deleterious materials in the No. 4 to 3/8-in. size range were substituted for the crushed limestone because a previous study by Sweet (1) has shown that deleterious particles passing a 3/8-in. screen have little effect on the freeze-thaw durability of concrete. All the coarse aggregate was vacuum saturated before mixing. The fine aggregate was not vacuum saturated, but was mixed with enough water to fill all surface-connected pores and left in this condition for 24 hr before mixing.

Mixing was accomplished by means of a modified food mixer with 1/4-cu ft capacity.

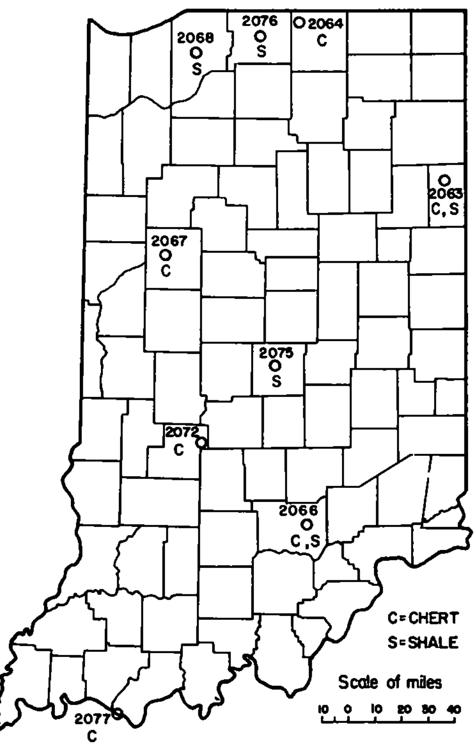

Figure 1. Map of Indiana locating sources of chert and shale gravels used in this study.

The concrete was molded into 3- by 4- by 16-in. beams, and these were cured by immersion in lime water for 13 days following removal of the specimens from the molds one day after casting.

Freeze-thaw testing of the beams was conducted according to the ASTM Method of Test for Resistance of Concrete Specimens to Rapid Freezing in Air and Thawing in Water, ASTM Designation C 291-57 T (2). The end point adopted for this series of tests was 50 percent relative dynamic modulus of elasticity of 300 cycles of freezing and thawing, whichever occurred first.

A durability factor was determined for each beam by the method shown in ASTM Designation C 291-57 T. These durability factors provided an index of the amount of deep-seated deterioration taking place in each beam.

During the freeze-thaw testing program, a number of popouts occurred on the surfaces of some of both the chert and shale beams. These popouts often occurred on beams which showed no deep-seated failure such as would be evidenced by low durability factors. Since these popouts were all found to be caused by failure of pieces of chert and shale during the freeze-thaw testing, this was further studied. During the freeze-thaw testing program, visual observation of any new popouts was made each time a beam was tested for its fundamental transverse frequency (that is, every 10-20 cycles of freezing and thawing). The position of each popout was noted as well as the approximate size of the piece of deleterious material causing it.

In order to compare the relative severity of surface deterioration of the beams, it was necessary to determine an index number for each beam which would give an indication of the relative popout damage suffered by that beam. An arbitrary numerical index based on sizes of the deleterious particles causing the popouts, number of popouts, and numbers of cycles at which the popouts occurred was developed and can be explained as follows:

$$SDF = \frac{s_1}{c_1} + \frac{s_2}{c_2} + \frac{s_3}{c_3} + \ldots\ldots\ldots + \frac{s_n}{c_n}$$

in which

SDF = surface durability factor for each beam;

s = size factor, s_1 for popouts caused by deleterious particles ⅜ to ½ in. in size, s_2 for popouts ½ to ¾ in. in size (average of 2 diameters); and

c = cycle factor, c_1 for cycles 1 to 100, c_2 for cycles 101 to 200, c_3 for cycles 201 to 300.

For beams whose relative moduli of elasticity dropped below 50 before 300 cycles of freezing and thawing were attained, this arbitrary equation does not result in an index that can be compared with beams undergoing the full 300 cycles. In many cases the beams that were removed from the freeze-thaw test before 300 cycles would have suffered additional surface deterioration if they had been allowed to reach 300 cycles. It is difficult to devise a correction factor which would satisfactorily eliminate this failing of the equation. However, only a few of the beams tested fall in this category, and these were especially noted in tabulating the data so that no direct comparisons would be made.

CHERT STUDIES

An experimental outline was set up in which three variables were introduced into the production of beams containing chert with all other controlled factors remaining constant. The three variables were as follows:

1. Source of chert—material from each of the six sources of chert from throughout the State of Indiana was used.

2. Specific gravity of chert—the chert from each of the six sources was separated into three groups based on bulk specific gravity (saturated surface-dry basis). The specific gravity ranges selected for these groups were 2.55 plus, 2.45-2.55, and 2.45

minus. Separation was accomplished using mixtures of carbon tetrachloride (specific gravity 1.58) and acetylene tetrabromide (specific gravity 2.97). Beams were made containing chert from each source and at each level of specific gravity.

3. Percentage of chert—chert from each combination of source and specific gravity group was included with the crushed stone coarse aggregate in the beams in amounts of 2, 4, and 10 percent, and a 6 percent level was included for chert from the 2.45 minus specific gravity group.

Statistical analysis of durability factors obtained from freeze-thaw studies of the chert beams (Table 1) indicated that the sources of the chert had no effect on resistance of concrete to deep-seated freeze-thaw deterioration. Even though the cherts were from different sources throughout Indiana, they resulted in nearly equal degrees of freeze-thaw deterioration when used in concrete in equal amounts having the same specific gravity ranges.

A definite difference in durability factors was found, however, for beams containing cherts of different specific gravity ranges. It was found that beams containing chert from the 2.45 minus specific gravity group had significantly lower durability than those containing chert from the 2.55 plus and 2.45-2.55 specific gravity ranges. This is in accord with the work of Sweet and Woods (3) which indicated low specific gravity chert to be the most susceptible to freeze-thaw deterioration.

The percentage of chert used also had a significant effect on the durability factors, but only at the 2.45 minus specific gravity level. At the 2.55 plus and 2.45-2.55 specific gravity levels, there was no significant difference among the durability factors of the beams containing different percentages of chert. Even 10 percent of chert from these specific gravity groups caused no deep-seated failure of beams in which it was included.

In the 2.45 minus specific gravity level the percentage of chert had a strong effect on durability of the concrete. Without exception, beams containing 10 percent of 2.45 minus chert suffered severe deep-seated deterioration. Every beam was intersected by at least one deep-seated crack caused by failure of the chert. The lowest durability factors recorded in the freeze-thaw testing occurred for this combination.

At the 6 percent level of the 2.45 minus specific gravity group, no deep-seated cracks occurred and the durability factors were found to be not significantly lower than those of the 2 and 4 percent levels of this gravity range. However, more variability in the data occurred at this level than at the 2 and 4 percent levels, that is, individual beams containing 6 percent of 2.45 minus chert had durability factors as low as 68.9 and 78.1 while others were as high as 97.2. The few low durability factors at the 6 percent level, while not nearly as low as those at the 10 percent level and not low enough to cause significant differences in the cell means, were low enough to suggest

TABLE 1

SUMMARY OF INDIVIDUAL DURABILITY FACTORS FOR FREEZE-THAW TESTING PROGRAM OF CONCRETE BEAMS CONTAINING SMALL PERCENTAGES OF CHERT COARSE AGGREGATE

Chert in Coarse Aggregate, (%)	Sp Gr Range, 2063 Chert			Sp Gr Range, 2064 Chert			Sp Gr Range, 2066 Chert			Sp Gr Range, 2067 Chert			Sp Gr Range, 2072 Chert			Sp Gr Range, 2077 Chert		
	2 55 Plus	2 45- 2 55	2 45 Minus	2 55 Plus	2 45- 2 55	2 45 Minus	2 55 Plus	2 45- 2 55	2 45 Minus	2 55 Plus	2 45- 2 55	2 45 Minus	2 55 Plus	2 45- 2 55	2 45 Minus	2 55 Plus	2 45- 2 55	2 45 Minus
2	97 5	98 2	94 1	98 2	98 2	96 4	98 8	97 3	89 1	97 9	96 9	96 1	98 2	97 0	93 8	98 2	98 2	94 4
	98 3	99 7	92 0	98 1	97 4	93 6	97 9	97 3	96 7	98 9	98 0	98 2	99 0	97 0	97 2	97 8	96 5	95 5
	97 9	99 0	93 1	98 2	97 8	95 0	98 4	97 3	92 9	98 4	97 5	97 2	98 6	97 0	95 5	98 0	97 4	95 0
4	99 8	98 3	88 9	98 2	97 3	95 5	98 0	96 5	95 5	97 7	96 4	95 5	99 0	94 6	96 5	97 2	97 3	90 3
	96 6	97 2	96 4	99 0	96 4	98 0	98 0	94 7	95 6	97 7	95 4	95 5	97 0	97 3	94 9	97 0	97 4	96 5
	98 2	97 8	92 7	98 6	96 9	96 8	98 0	95 6	95 6	97 7	95 9	95 5	98 0	96 0	95 7	97 1	97 4	93 4
6			96 3			94 7			88 7			84 5			92 1			87 8
			78 1			97 2			89 7			95 6			90 0			68 9
			87 2			96 0			89 2			90 1			91 1			78 4
10	98 2	71 0	24 4	99 1	82 5	11 2	97 0	93 5	29 6	99 1	83 2	57 3	98 9	92 0	64 7	97 9	96 3	30 0
	98 2	94 7	29 5	96 4	98 0	25 0	98 0	95 5	46 5	97 3	94 6	38 4	98 9	91 9	38 3	97 2	92 9	26 4
	98 2	82 9	27 0	97 8	90 3	18 1	97 5	94 5	38 1	98 2	88 9	47 9	98 9	92 0	51 5	97 6	94 6	28 2

that some deterioration can occur in concrete containing as little as 6 percent chert with a bulk specific gravity of less than 2.45.

Durability factors for beams containing 2 and 4 percent of 2.45 minus chert were shown by analysis of variance to be significantly lower than durability factors for beams containing the same percentages of chert from the two heavier specific gravity ranges. However, all the durability factors for these beams are high enough to indicate little deep-seated deterioration (Table 1). For example, the lowest durability factor computed for the 4 percent level of the 2.45 minus specific gravity range was 88.9 and the lowest cell mean was 92.7. These values are high enough to be considered indicative of sound concrete.

In summary, it appears that otherwise sound concrete containing up to 4 percent chert with a bulk specific gravity (saturated surface-dry basis) of 2.45 or less, or as much as 10 percent chert with a specific gravity greater than 2.45, can successfully withstand laboratory freeze-thaw exposure without undergoing deep-seated deterioration.

The effect of size of the individual chert particles on their freeze-thaw durability in concrete is of interest. Previous studies of deleterious substances have indicated a relationship between size of unconfined particle and lack of freeze-thaw durability. Wray and Lichtefeld (4) found in their study of Missouri cherts that saturated 1- to 1½-in. particles had less resistance to freezing and thawing failure than saturated ¾- to 1-in. particles. Thomas (5) saturated prisms of different sizes from the same rock and found that damage was greater the larger the specimen.

The effect of size on the durability of particles of deleterious materials in concrete is not so clear, however. Sweet and Woods (3) embedded saturated chert pieces of three sizes, ¾ to 1 in., ½ to ¾ in., and ⅜ to ½ in., in 1-in. mortar cubes and subjected these cubes to up to 309 cycles of freezing and thawing. They found that the cubes failed at an earlier cycle for the ¾- to 1-in. pieces than for the ½- to ¾-in. pieces, and that no failure occurred in the cubes containing the ⅜- to ½-in. pieces. Klieger (6), however, found no apparent relationship between size of unsound aggregate particles and durability as long as the air content of the mortar was held constant. Walker and McLaughlin (7) demonstrated that lightweight chert less than ⅜ in. in size would not cause deep-seated freeze-thaw deterioration in concrete, but their method of test did not distinguish between the degrees of resistance to freeze-thaw deterioration exhibited by different sizes of chert larger than ⅜ in.

In this study no attempt was made to determine the effect of size on durability. However, for those beams that had suffered deep-seated cracking as a result of freezing and thawing, a qualitative study was conducted to determine the sizes of the pieces of chert intersected by each crack. In each case it appeared that the ¾- to 1-in. piece had provided most of the disruptive force. In no case was a crack caused by ½- to ¾- or ⅜- to ½-in. pieces alone. This does not mean that ½- to ¾- or ⅜- to ½-in. pieces could not cause deep-seated failure of concrete, but does show that they are not as harmful as the larger pieces. Larger chert particles in concrete appear to have less resistance to freeze-thaw deterioration than smaller ones.

Although freeze-thaw testing of concrete specimens is primarily intended to cause deep-seated failure and subsequent loss of strength of concrete specimens containing unsound aggregates, surface deterioration of the concrete, which in some cases is equally as important as deep-seated failure, often occurs in these tests. A part of this study was to determine how surface deterioration is influenced by each of the variables introduced into the freeze-thaw study.

Surface deterioration factors for the chert beams are given in Table 2. These data indicate no significant difference in severity of popout damage among the six chert sources. The major differences in severity of surface deterioration apparently were caused by material from different bulk specific gravity ranges. For all six cherts a negligible amount of surface deterioration occurred in beams containing material from the 2.55 plus and 2.45-2.55 specific gravity ranges. Material from the 2.45 minus gravity range, however, resulted in a significant amount of popout damage in beams made from each of the six cherts. In general, the 6 to 10 percent levels within

TABLE 2

SUMMARY OF SURFACE DETERIORATION FACTORS FOR FREEZE-THAW TESTING PROGRAM OF CONCRETE BEAMS CONTAINING SMALL PERCENTAGES OF CHERT COARSE AGGREGATE

Chert in Coarse Aggregate, (%)	Sp Gr Range, 2063 Chert			Sp Gr Range, 2064 Chert			Sp Gr Range, 2066 Chert			Sp Gr Range, 2067 Chert			Sp Gr Range, 2072 Chert			Sp Gr Range, 2077 Chert		
	2 55 Plus	2 45-2 55	2 45 Minus	2 55 Plus	2 45-2 55	2 45 Minus	2 55 Plus	2 45-2.55	2 45 Minus	2 55 Plus	2 45-2 55	2 45 Minus	2 55 Plus	2 45-2 55	2 45 Minus	2 55 Plus	2 45-2 55	2 45 Minus
2	0 0	0.0	0 6	0 0	0 6	0 0	0 0	0 0	3.0	0 0	0 0	1 0	0 0	1 0	0 0	0 0	0 0	0 0
	0 0	0 0	0 0	0 0	0 0	0 3	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0
	0 0	0 0	0 3	0 0	0 3	0 2	0 0	0 0	1 5	0 0	0 0	0 5	0 0	0 5	0 0	0 0	0 0	0 0
4	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 6	0 0	0 0	0 0	1 0	0 0	0 0	0 0	0 0	0 0	1 7
	0 0	0 0	1 3	0 0	0 3	1 0	0 0	0 0	1 2	0 0	0 0	0 0	0 0	0.0	0.0	0 0	0 0	2 0
	0 0	0 0	0 7	0 0	0 2	0 5	0 0	0 3	0 6	0 0	0 0	0 5	0.0	0 0	0 0	0.0	0 0	1 9
6			1 0			2 6			1 7			0 3			1 7			0 3
			0 5			4 3			1 3			0 0			0 0			0 0
			0 8			3 5			1 5			0 2			0 9			0 2
10	0 0	0 0	0 0[1]	0 0	0 0	0 0[1]	0.0	0 0	2 5[1]	0 0	0 0	0.3	0 0	0 0	3 0	0 0	0 0	1 5[1]
	0 0	0 6	0 3[1]	0 0	0 0	0 0[1]	0 0	0 0	2 5[1]	0 0	0 0	0 0[1]	0.0	0 3	1 7[1]	0.0	0 0	0 0[1]
	0 0	0 3	0 2	0 0	0 0	0 0	0 0	0 0	2 5	0 0	0 0	0 2	0 0	0 2	2 4	0 0	0 0	0 8

[1]Removed from freeze-thaw test at less than 300 cycles

the 2.45 minus specific gravity range had higher surface deterioration factors than the 2 and 4 percent levels, but this was not true for all six cherts. For example, in the 2.45 minus specific gravity range for chert 2077, the 4 percent level had an average surface deterioration factor of 1.9, while the 6 percent level had an average factor of only 0.2. This seeming anomaly is probably due more to random positioning of the pieces of chert than to any error in procedure or real differences in the material.

It also should be noted that, in most cases, larger factors were obtained for beams in the 6 percent level of the 2.45 minus specific gravity range than in the 10 percent level. This was primarily due to failure of most of the 10 percent beams to complete a full 300 cycles of freezing and thawing while all the 6 percent beams lasted the full 300 cycles. Thus the 10 percent specimens were not exposed to as many cycles of freezing and thawing as those containing 6 percent chert.

In summary, surface deterioration in the beams containing chert paralleled the deep-seated failure of these beams. In both cases the different sources had little, if any, effect. For all sources failure occurred primarily in the 6 and 10 percent levels of the 2.45 minus specific gravity range.

SHALE STUDIES

An experimental outline was formulated in which two variables were introduced into design of the concrete beams containing shale with all controlled factors remaining constant. This experimental design differed from that of the chert study in that no specific gravity separation of the shale was made. It was set up as a two-way crossed classification. The two variables in the design were as follows:

1. Source of shale—material from each of the sources was blended with the crushed stone coarse aggregate in different beams.
2. Percentage of shale—shale from each of the sources was combined with the crushed stone coarse aggregate in blends of 2, 4, 6, and 10 percent.

Study of the durability factors for concrete beams containing shale (Table 3) indicates that no combination of sources and percentages (up to 10 percent) of shale resulted in deep-seated failure of the beams. Only a few beams were found to have durability factors below 90, and these few values appear to be well distributed throughout the data. Only one cell mean is below 90 and this is at the 4 percent level, while the durability factors at the 6 and 10 percent levels for this same shale (2066) are well above 90. This indicates that the low mean for the 4 percent level is probably due to random error.

TABLE 3

SUMMARY OF INDIVIDUAL DURABILITY FACTORS FOR FREEZE-THAW TESTING PROGRAM OF CONCRETE BEAMS CONTAINING SMALL PERCENTAGES OF SHALE COARSE AGGREGATE

Shale in Coarse Aggregate (%)	Shale Source				
	2063	2066	2068	2075	2076
2	100.0	97.8	99.5	96.5	96.0
	98.8	96.0	85.5	-	96.5
	92.0	92.8	94.0	88.5	96.0
	-	94.5	-	97.2	95.4
	-	95.3	-	97.2	95.8
	-	-	-	98.2	96.3
	97.0	95.3	93.0	95.5	96.0
4	96.3	96.5	94.3	96.4	96.3
	85.3	-	-	96.4	94.0
	97.1	90.8	-	97.2	97.2
	-	80.5	97.1	-	93.0
	-	-	96.5	-	96.0
	-	87.2	95.3	-	99.5
	92.9	88.8	95.8	96.7	96.0
6	99.0	97.4	-	98.5	99.8
	98.2	92.5	98.4	87.0	89.0
	96.0	98.2	-	96.5	100.0
	99.0	96.3	87.9	-	97.0
	98.7	87.6	85.1	-	89.0
	100.0	96.5	-	-	88.4
	98.5	94.8	90.5	94.0	93.9
10	95.5	87.0	-	-	90.8
	86.5	96.5	83.0	98.0	95.4
	95.0	-	-	96.2	95.5
	-	97.0	95.3	96.4	-
	-	96.0	93.5	98.2	-
	-	96.2	91.6	96.4	-
	92.3	94.5	90.9	97.0	93.9

Thus it appears from these data that in amounts of up to 10 percent, little difference in resistance of the concrete to deep-seated deterioration was caused by the different shales even though they were from widely separated areas throughout the state and had significantly different basic properties. This is in accord with the findings of Lang (8) who noted that for pavement concrete containing small percentages of shale, the only harmful effect of the shale due to freezing and thawing consisted of surface deterioration of the concrete.

A comparison of durability factor data for the cherts and shales shows that the only deep-seated deterioration caused by either of these materials was due to chert with a bulk specific gravity (saturated surface-dry basis) of less than 2.45. Since some of the shale samples contained a considerable quantity of material which is of low bulk specific gravity even for shale (2.15 minus or 2.25 minus), and since none of these shales produced any deep-seated failure, specific gravity apparently does not have the same relationship to resistance to deep-seated failure for shales as it does for cherts.

Surface deterioration factors for the shale beams are given in Table 4. It is evident

TABLE 4

SUMMARY OF SURFACE DETERIORATION FACTORS FOR FREEZE-THAW TESTING PROGRAM OF CONCRETE BEAMS CONTAINING SMALL PERCENTAGES OF SHALE COARSE AGGREGATE

Shale in Coarse Aggregate (%)	Shale Source				
	2063	2066	2068	2075	2076
2	0.0	0.0	2.0	0.0	0.0
	0.0	0.0	2.0	-	0.0
	0.0	0.0	2.0	0.0	0.0
	-	0.0	-	0.0	0.0
	-	0.0		0.0	0.0
	-	-	-	0.0	0.0
	0.0	0.0	2.0	0.0	0.0
4	0.0	0.0	0.0	0.0	0.0
	0.0	-	-	0.0	0.0
	0.0	0.0	-	0.0	0.0
	-	0.0	6.0	-	0.0
	-	-	0.3	-	0.0
	-	0.0	1.7	-	0.0
	0.0	0.0	2.0	0.0	0.0
6	0.0	0.0	-	0.0	1.0
	0.0	0.5	4.2	0.0	0.0
	0.0	0.3	-	1.0	1.0
	2.0	2.0	0.3	-	0.0
	0.0	0.0	2.5	-	0.0
	2.0	0.0	-	-	1.0
	0.7	0.5	2.3	0.3	0.5
10	0.0	0.5	-	-	4.0
	0.0	1.0	1.2	0.0	3.3
	0.0	-	-	1.0	0.3
	-	0.0	5.9	0.0	-
	-	0.0	3.0	1.0	-
	-	0.0	8.5	0.0	-
	0.0	0.3	4.7	0.4	2.5

from these data that shale 2068 caused considerably more popout damage than any of the other shales. Beams containing shale 2068 had higher surface deterioration factors than beams containing the other shales at every percentage level, and shale 2068 was the only shale to cause even a single popout in beams containing 2 to 4 percent of this material.

Only a small difference in performance relative to popout damage could be detected among the other four shales. No surface deterioration occurred for any of these shales when used in amounts up to and including 4 percent. At the 6 to 10 percent levels, shales 2066 and 2076 caused little more popout damage than shales 2063 and 2075, but the difference is slight.

Comparison of surface deterioration factors for the cherts and shales indicates that, except for sample 2068, the shales caused about the same amount of surface deterioration as cherts of the 2.45-2.55 specific gravity range. In general, the shales caused greater popout damage than cherts of the 2.55 plus specific gravity range and lesser damage than cherts of the 2.45 minus gravity range. Shale 2068, however, resulted in more severe surface deterioration than any of the cherts of any specific gravity range.

COMPARISON OF AIR VOID PARAMETERS AND DURABILITY OF CONCRETE MIXED BY HAND AND MACHINE

During the early stages of the freeze-thaw study of concrete beams containing shales, a few beams were prepared from hand-mixed concrete to see how their durabilities would compare with those of the regular beams from machine-mixed concrete used in the freeze-thaw testing program. It was found that in nearly all cases the hand-mixed concrete had lower durability factors than machine-mixed concrete of identical composition. It was felt that this presented an opportunity to study the air-void parameters of these two classes of concrete in an attempt to explain the observed differences.

By means of the linear traverse technique outlined by Fears (9), the total percentage of entrained and entrapped air was determined for each beam. Using the values obtained for total percentage of air and voids per inch of traverse, it was possible to compute specific surface areas and bubble spacing factors for the beams using Powers method (10). Powers theorized that the increase in durability afforded concrete by means of air entrainment is largely a function of the spacing of the air voids in the concrete. He suggested that a concrete containing air voids with high specific surface area, and thus with a low void spacing factor, would receive more protection from the air voids than one containing air bubbles having a low specific surface area and a high spacing factor. He considered a void spacing of about 0.01 in. to be critical; those concretes with spacing factors lower than 0.01 in. were thought to be well protected from freezing and thawing deterioration; those with spacing factors greater than 0.01 in. were thought to be poorly protected.

This theory is supported by the results of the linear traverse studies (Table 4A and Figure 2). For the 14 beams studied, it was found that those having high durability factors (83 and above) all had spacing factors below 0.01 and those with low durability factors (21 and below) had spacing factors above 0.01. Also, the specific surface areas of the beams with high durability factors were all higher than those with low durability factors.

INFLUENCE OF BASIC PROPERTIES OF THE CHERTS AND SHALES ON FREEZE-THAW DURABILITY

The primary objective of this portion of the study was to study the basic properties of cherts and shales in Indiana's gravel aggregates and to determine how these properties

TABLE 4A

RESULTS OF AIR VOID STUDIES OF CERTAIN CONCRETE BEAMS BY MEANS OF THE LINEAR TRAVERSE TECHNIQUE

Beam No	Beam Description	Durability Factor	% Air	Voids per In	Calculated Specific Surface Area of Voids (sq in /cu in)	Calculated Void Spacing Factor (in)
S6-3	4% shale No 2063, machine mixed	97 1	3 3	4.2	506	0 0080
S6-6	4% shale No 2063, hand mixed	21 1	4 1	2.9	287	0.0131
S8-6	10% shale No 2063, machine mixed	95 0	3 8	4 8	512	0 0075
S8-1	10% shale No 2063, hand mixed	6 0	3.8	2.0	208	0 0185
S9-5	10% shale No. 2068, machine mixed	83 0	2.7	3 5	523	0 0088
S9-1	10% shale No 2068, hand mixed	8 3	3 3	2 7	324	0 0126
S20-2	10% shale No 2066, machine mixed	96 5	3.5	3 8	441	0 0091
S20-5	10% shale No 2066, hand mixed	14 2	3.0	2.0	269	0 0165
S21-2	6% shale No 2075, machine mixed	72 4	3 0	3.8	512	0.0083
S21-6	6% shale No. 2075, hand mixed	10 6	3 1	2 1	273	0 0154
S22-1	10% shale No 2076, machine mixed	90 8	2 8	3 2	451	0 0098
S22-4	10% shale No 2076, hand mixed	30.1	2 7	2 2	326	0.0139
S23-3	10% shale No 2075, machine mixed	96 2	3 1	5 0	637	0 0066
S23-4	10% shale No 2075, hand mixed	5 3	3 8	3 0	316	0 0121

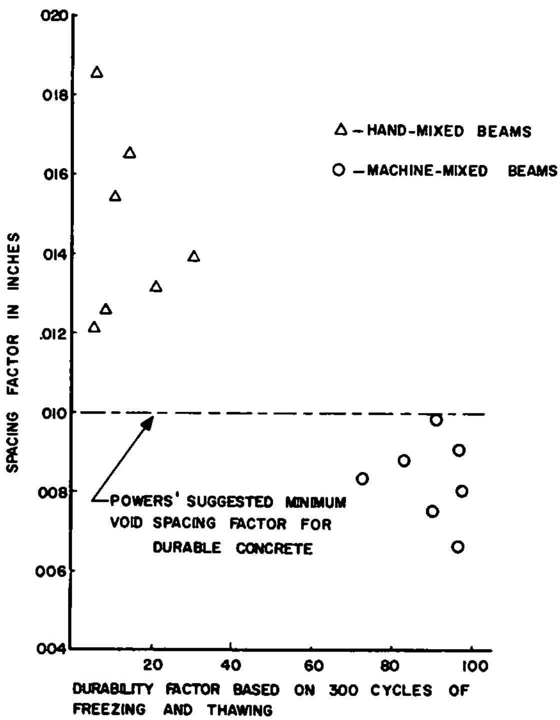

Figure 2. Relationship of durability factors to void spacing factors for air-entrained concrete.

affect the freeze-thaw durability of these materials. The properties discussed are porosity, absorption, mineralogy, texture and microstructure.

Porosity

Much work relating porosity and durability of crushed limestone aggregates has been done by Sweet (11) and Fears (12). Early studies by Cantrill and Campbell (13), Wuerpel and Rexford (14), and Sweet and Woods (3) correlated porosity and durability for cherts. However, little correlation of this type has been attempted for shales, and the chert studies mentioned were made before air-entrained concrete had come into use and before freeze-thaw testing had reached its present state of development. In addition, little was done in a quantitative sense in these previous studies. For example, it was determined in a qualitative way that lightweight chert causes deterioration when used in concrete exposed to freezing and thawing, but nothing has been done to determine the quantity of this material required to cause deterioration.

Since porosity is so important in the freezing and thawing durability of concrete aggregates, several studies of voids in the cherts and shales were made. Total volume of voids and volume of voids less than and greater than 5 microns in diameter were determined for the three specific gravity groups of cherts 2067 and 2077, and total porosity was determined for the five shale samples.

Total Porosity

Total porosity was calculated by means of the following relationship between bulk and true specific gravity:

$$n = \frac{V_v}{V} = 1 - \frac{S_b}{S_t}$$

in which

- n = porosity,
- v = total volume,
- V_v = volume of voids,
- S_t = true specific gravity, and
- S_b = bulk specific gravity.

The total porosity of chert is inversely related to its bulk specific gravity (a more easily measured characteristic than porosity) for materials of the same true specific gravity, and as such is generally reflected in specifications for chert in concrete aggregate. It has been found that the most porous cherts (those with the lowest bulk specific gravities) cause the most severe freeze-thaw deterioration.

Freeze-thaw studies of concrete beams containing chert showed that significant deep-seated and surface deterioration took place only in beams containing 6 to 10 percent of material from the 2.45 minus specific gravity group. For the samples tested, the

TABLE 5

TOTAL POROSITIES OF CHERT SAMPLES

Source	Gravity Group (by heavy liquid separation)	Bulk Sp Gr	True Sp Gr	$\frac{S_b}{S_t}$ *	Porosity, n (%)
2067	2.55 plus	2.56	2.64	0.970	3.0
	2.45-2.55	2.44	2.64	0.924	7.6
	2.45 minus	2.30	2.64	0.871	12.9
2077	2.55 plus	2.56	2.64	0.970	3.0
	2.45-2.55	2.47	2.64	0.936	6.4
	2.45 minus	2.31	2.65	0.872	12.8

*$n = (1 - \frac{S_b}{S_t})\ 100$

porosity of chert in this specific gravity group was nearly 13 percent (Table 5). Chert from the 2.45-2.55 and 2.55 plus specific gravity groups, which caused little freeze-thaw deterioration, had porosities of only about 7 and 3 percent, respectively. This relationship definitely supports the ideas of Wuerpel and Rexford (14) and Sweet and Woods (3) that cherts with high porosity are more susceptible to freeze-thaw deterioration than those with low porosity, and further demonstrates that this concept holds for air-entrained concrete as well as concrete with no entrained air. It also suggests that the 2.45 bulk specific gravity level (saturated surface-dry basis) suggested by Sweet and Woods as the critical level of separation between unsound chert and durable chert is realistic even for air-entrained concrete.

The lack of protection afforded porous aggregates such as these lightweight cherts by air-entrained cement paste has been explained in general terms by Powers (15). When saturated aggregate particles surrounded by air-entrained cement paste are subjected to freezing, the water in the paste is able to move to the "escape boundaries" provided by entrained bubbles in the paste, and no excess hydraulic pressures are able to develop. Thus the paste itself is protected from dilation. However, saturated porous rock particles enclosed by the paste still perform as virtually enclosed containers and are only a little better off than if the paste were not air-entrained. Probably the paste bubbles near the contact between aggregate particle and paste do accept a small amount of the excess water produced by freezing the saturated aggregate, but for saturated aggregates of high porosity, the amount of excess water is too large to be taken on by the bubbles in the paste immediately adjacent to the aggregate. For this reason, protecting the paste by air entrainment, while possibly successful for aggregates of low porosity, fails to protect saturated highly porous aggregate particles. This concept helps to explain the findings of Axon, Willis, and Reagel (16) who noted that the entrainment of air resulted in a definite improvement in durability of concrete containing limestones with good service records, but only caused a slight increase in durability for concrete made with chert-rich aggregate with a fair service record, and affected no appreciable improvement in durability of concrete made with chert-rich aggregate with a poor service record.

As given in Table 6, the porosities of the different shales varied widely. For example, shale 2068, which was the softest and weakest of the shales, was nearly twice as porous as any of the other shales, and over five times as porous as shale 2063, the least porous and the most-indurated of the samples. The widely varying porosities of the shales had no effect on the amount of deep-seated freeze-thaw detioration caused by these materials, however. In amounts up to 10 percent, none of the shales caused any deep-seated damage to the concrete in which they were used. This lack of deep-seated deterioration of concrete containing shales of relatively high porosity was due

TABLE 6

TOTAL POROSITIES OF SHALE SAMPLES

Source	Bulk Sp Gr	True Sp Gr	$\frac{S_b}{S_t}$*	Porosity, n (%)
2063	2.28	2.38	0.958	4.2
2066	2.06	2.39	0.862	13.8
2068	2.00	2.58	0.775	22.5
2075	2.24	2.45	0.914	8.6
2076	2.08	2.47	0.842	15.8

$^{*}n = (1 - \frac{S_b}{S_t})\ 100$

to the inherent structural weakness of these materials. Since the shale is considerably weaker than the surrounding mortar, it will fail internally due to the pressures developed in freezing rather than disrupt the mortar. This was demonstrated by comparison between pieces of chert and shale that had failed in the freeze-thaw test and subsequently had been removed from the beams. The chert pieces, which had caused deep-seated deterioration had broken into many pieces, but the individual pieces were still relatively hard and firm. The shales, on the other hand, had disintegrated into weak crumbly masses although they had caused no deep-seated deterioration. From all appearances these shales had failed internally before the pressures due to freezing could develop enough to break the surrounding mortar.

Where an individual shale particle occurred close to the surface of a beam, the enclosing mortar layer was often not strong enough to resist the hydrostatic pressures developed by freezing the saturated particle. In this case the mortar was disrupted, resulting in surface deterioration in the form of a popout or pit. The relative porosities of the shales had a marked relationship to severity of surface deterioration. Shale 2068, the material having the highest porosity of the group (Table 6), caused considerably more popout damage than any of the other shales or any of the cherts. It appears that this larger amount of deterioration is related to the greater porosity of shale 2068, but, as will be discussed later, other factors such as the size of the pores and the amount of absorption of the shale probably have a greater effect on the durability of the aggregate than the total porosity.

It should be noted that among the other four shales the relationship between porosity and surface deterioration is not so clear. Shales 2076 and 2066 are considerably more porous and more absorptive than 2063 and 2075, yet caused only a little more popout damage than 2063 and 2075

In summary of the relationship of total porosity to freeze-thaw deterioration of concrete containing cherts and shales, the following points should be brought out:

1. Although other pore characteristics such as pore size and absorptivity may have a strong influence on the resistance of the cherts tested to both deep-seated and surface deterioration, there is a definite relationship between total porosity and the freeze-thaw resistance of these materials. The more porous fractions from all six chert groups caused more freeze-thaw deterioration than the less porous material.

2. Total porosity of shales was related to severity of surface deterioration of concrete in which the shales were used, but in spite of widely varying porosities, none of the shales resulted in deep-seated deterioration of concrete in which they were used in amounts up to ten percent. Shale 2068, which was considerably more porous than the other shales, caused much more surface deterioration than the others but caused no deep-seated deterioration.

Size of Pores

Although recognizing a relationship between total porosity and freeze-thaw durability of aggregates, Sweet (1) and Fears (12) have contended that the durability of aggregates is dependent more on the size and continuity of aggregate pores than on total porosity. Lewis and Dolch (17) maintained that the harmful pore size is that large enough to permit water readily to enter much of the pore space but not large enough to permit easy drainage. Studies by Sweet (1), and Fears (12) have indicated that critical pore size for freezing and thawing durability of limestone aggregates is about 5 microns. Blanks (18) has shown 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.

These previous investigations indicate the importance of microvoids (pores less than 5 microns in diameter) in the durability of aggregates. Therefore, part of the present study was devoted to determining if this relationship between pore size and durability holds for Indiana cherts. For cherts 2067 and 2077, the percentage of total volume of aggregate occupied by voids greater than 5 microns in diameter was determined by a linear traverse study of polished surfaces and this percentage was subtracted from the total porosity to obtain the percentage of total aggregate volume occupied by microvoids. (As used here, the term "microvoids" refers to voids less than 5 microns in diameter.) The linear traverse technique used was similar to that reported by Fears (9) for study of air-voids in hardened concrete. Recording of traverse lengths was accomplished by means of a Hunt-Wentworth recording micrometer of the type commonly used for micrometric mineralogical analyses.

The results of the pore size studies are given in Table 7. It was found that the volume of microvoids was somewhat less than expected. Sweet (1) 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. If Sweet's criteria were to be applied to the chert fractions whose void ratios are given in Table 7, it would seem that none of the material in these fractions would be susceptible to freeze-thaw deterioration since none of this material has microvoid ratios (as used here the term "microvoid ratio" refers to the ratio of volume of voids less than five microns in diameter to the bulk volume of the aggregate) as high as 0.091. The highest ratio, 0.064 for the 2.45 minus specific gravity group of chert 2067, is only slightly higher than the 0.057 ratio designated as the upper limit for aggregates with good service records. In spite of these relatively low microvoid ratios, the 2.45 minus specific gravity chert from sources 2067 and 2077 caused serious freeze-thaw deterioration in concrete in which it comprised 10 percent of the coarse aggregate.

It also should be noted that practically no freeze-thaw deterioration occurred in concrete containing chert from the 2.45-2.55 specific gravity range even though this material contained nearly as large a ratio of microvoids as did chert from the 2.45 minus specific gravity group. In spite of the lack of difference in volume of microvoids between the 2.45-2.55 and 2.45 minus specific gravity ranges, there is considerable difference in total porosity between these ranges. As given in Table 7, the high total porosity of the 2.45 minus chert as compared to the 2.45-2.55 material is primarily

TABLE 7

RELATION OF PORE SIZE TO DEGREE OF SATURATION FOR CHERTS 2067 AND 2077

Source	Sp Gr Range	Total Porosity (%)	% of Bulk Volume Consisting of Voids >5 Microns in Diameter	% of Voids Volume Consisting of Voids >5 Microns in Diameter	% of Bulk Volume Consisting of Voids <5 Microns in Diameter	% of Voids Volume Consisting of Voids <5 Microns in Diameter	Degree of Saturation (%)
2067	2 55 plus	3 0	0 6	20.0	2 4	80 0	82 3
	2 45-2 55	7 6	1.9	25 0	5 7	75 0	92 5
	2 45 minus	12 9	6 5	50 4	6.4	49 6	100 0
2077	2.55 plus	3 0	0.4	13 3	2 6	86 7	78.3
	2.45-2.55	6.4	2 3	35 9	4 1	64.1	87 7
	2 45 minus	12.8	7 9	61.7	4 9	38 3	90 5

due to the increase in voids larger than 5 microns in diameter in the 2.45 minus material. As the total porosity of the chert increases in going from material of high to low bulk specific gravity, the voids larger than 5 microns in diameter constitute an increasingly larger percentage of the total pore space, and conversely, the microvoids make up an increasingly smaller percentage of the pore space (Table 7). For example, for chert 2067, the microvoids constitute 80 percent of the total pore space in the 2.55 plus chert, 75 percent in the 2.45-2.55 material, and only 50 percent in the 2.45 minus range.

On the basis of this study, it appears the microvoids ratio is not as reliable an indicator of freeze-thaw durability of chert aggregate as Sweet (1) and Fears (12) found it to be for limestone aggregate. The results of this study indicate that some other pore characteristic or, more probably, a combination of characteristics (perhaps including volume of microvoids) is probably the main factor in determining chert durability.

As given in Table 7, the degree of saturation of the cherts increases with increasing total porosity and decreasing bulk specific gravity. This increase is probably caused by the larger percentage of voids larger than 5 microns in diameter in the more porous material. Under conditions of vacuum saturation and frequently repeated immersion as used in these tests, a piece of aggregate containing numerous large voids as well as microvoids would probably reach a high degree of saturation more readily than a particle containing only microvoids, because of the greater ease of flow through the larger voids. In this way it is thought that 2.45 minus chert reaches a higher degree of saturation than heavier less porous fractions, and that this high degree of saturation is a prime factor in the lack of durability of the 2.45 minus material.

Besides being a factor in the permeability of the chert, the size of the pores undoubtedly determines whether dilation occurs. Pores or bulges in pores that are large enough to act as escape boundaries for the water under hydrostatic pressure (15) will cause no dilation. Obviously some of the large pores in the 2.45 minus material are in this category. The critical size between pores which will cause dilation and those which will act as escape boundaries depends on the length and tortuosity of the pores. It is possible that dilation in lightweight cherts occurs entirely within voids less than 5 microns in diameter which are supplied with water by the larger voids, but it is quite probable that some of the voids larger than 5 microns are too small to serve as escape boundaries and thus contribute to dilation of the particle.

In summary, the microvoids ratio does not provide a satisfactory indication of the freeze-thaw durability of chert. Instead chert durability is probably more closely related to the degree of saturation of these small voids (and larger voids that are too small to act as escape boundaries) which is strongly influenced by the presence of voids greater than 5 microns in diameter. Based on this concept and the results of the freeze-thaw tests, it appears that total porosity, as reflected in bulk specific gravity, serves as a satisfactory criterion for predicting the freeze-thaw durability of chert.

ABSORPTION

Lewis and Dolch (17) have stated that, "The lack of durability of an aggregate in freezing and thawing is primarily dependent on its ability to become and stay highly saturated under the given conditions of moisture." Thus besides being porous, an aggregate must be absorptive in order to be susceptible to freeze-thaw deterioration. This section discusses the relationship between (a) vacuum-saturated absorption, and (b) rate of absorption of cherts and shales, and the resistance of these materials to freeze-thaw deterioration.

Vacuum-Saturated Absorption

Comparison of the results of the vacuum-saturated absorption tests of the chert groups (Table 8) to the results of freeze-thaw tests of concrete containing the cherts indicates a direct relationship between percentage of absorption and lack of freeze-thaw durability for chert. Cherts of the 2.45 minus specific gravity ranges for all six sources had absorption percentages about twice as great at those for the 2.45-2.55 groups and five times as great as the 2.55 plus groups. The absorptions of these materials are directly related to their porosities. As given in Table 7, the higher percentage of pores larger than 5 microns in diameter found in the highly porous 2.45 minus material as compared to that in the 2.45-2.55 and 2.55 plus fractions apparently facilitated absorption in the lightweight chert with a resulting higher degree of saturation than could be obtained in the heavier materials.

As has been shown previously, chert with a specific gravity of less than 2.45 caused the only deterioration in the freeze-thaw test. From the data it appears that cherts with vacuum-saturated adsorptions of less than about 3 percent will not cause significant freeze-thaw deterioration when included in concrete in amounts up to 10 percent of the coarse aggregate for the size groups studied. 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.

The vacuum-saturated absorption values for the shales (Table 9) varied considerably—the percentages of absorption roughly paralleling the porosities of the shale samples. As was the case for porosity, the absorption of the shales had no apparent influence on the resistance of concrete containing these materials to deep-seated deterioration. This is shown by the fact that none of the shales, including the most absorptive samples, caused any deep-seated freeze-thaw failure when used in concrete in amounts up to 10 percent of the coarse aggregate.

There does appear to be a relationship between absorption and severity of surface deterioration, however. Shale 2068, which has the greatest absorption, caused by far the greatest amount of popout damage. As was the case for porosity, the influence of absorption on surface deterioration is not so distinct among the other four shales. Shales 2066 and 2076 had considerably greater absorptions than shales 2063 and 2075 (Table 9), but there was little difference in the amount of surfact deterioration caused by these four shales. Although the popout damage caused by shales 2066 and 2076 was slightly

TABLE 8

VACUUM-SATURATED ABSORPTION VALUES FOR CHERT SAMPLES

Source	Saturated Surface-Dry Bulk Specific Gravity Group	Size Range (in.)	Absorption (%)
2063	2.55 plus	3/4-1	1.26
		1/2-3/4	1.21
		3/8-1/2	1.12
			1.20
	2.45-2.55	3/4-1	3.02
		1/2-3/4	2.84
		3/8-1/2	3.30
			3.05
	2.45 minus	3/4-1	6.26
		1/2-3/4	6.13
		3/8-1/2	6.88
			6.42
2064	2.55 plus	3/4-1	0.82
		1/2-3/4	0.93
		3/8-1/2	1.00
			0.92
	2.45-2.55	3/4-1	2.86
		1/2-3/4	2.78
		3/8-1/2	2.90
			2.85
	2.45 minus	3/4-1	5.71
		1/2-3/4	5.56
		3/8-1/2	5.62
			5.63
2066	2.55 plus	3/4-1	1.14
		1/2-3/4	1.30
		3/8-1/2	1.09
			1.18
	2.45-2.55	3/4-1	2.76
		1/2-3/4	3.12
		3/8-1/2	2.86
			2.91
	2.45 minus	3/4-1	6.12
		1/2-3/4	6.36
		3/8-1/2	6.11
			6.20
2067	2.55 plus	3/4-1	1.09
		1/2-3/4	1.05
		3/8-1/2	1.19
			1.11
	2.45-2.55	3/4-1	2.96
		1/2-3/4	2.84
		3/8-1/2	2.67
			2.82
	2.45 minus	3/4-1	5.58
		1/2-3/4	5.59
		3/8-1/2	5.62
			5.60
2072	2.55 plus	3/4-1	1.02
		1/2-3/4	1.11
		3/8-1/2	1.11
			1.08
	2.45-2.55	3/4-1	2.55
		1/2-3/4	2.80
		3/8-1/2	2.96
			2.77
	2.45 minus	3/4-1	5.26
		1/2-3/4	5.39
		3/8-1/2	5.91
			5.52
2077	2.55 plus	3/4-1	1.06
		1/2-3/4	0.92
		3/8-1/2	1.04
			1.01
	2.45-2.55	3/4-1	2.31
		1/2-3/4	2.69
		3/8-1/2	2.27
			2.42
	2.45 minus	3/4-1	4.77
		1/2-3/4	4.20
		3/8-1/2	4.51
			4.49

TABLE 9

VACUUM-SATURATED ABSORPTION VALUES FOR SHALE SAMPLES

Source	Size Range (in.)	Absorption (%)
2063	¾-1	1.54
	½-¾	1.60
	⅜-½	2.17
		1.77
2066	¾-1	7.07
	½-¾	7.05
	⅜-½	7.04
		7.05
2068	¾-1	12.83
	½-¾	12.23
	⅜-½	12.63
		12.56
2075	¾-1	3.64
	½-¾	4.06
	⅜-½	4.68
		4.13
2076	¾-1	7.82
	½-¾	8.61
	⅜-½	7.91
		8.11

greater than that caused by shales 2063 and 2075, it was not as great as might be expected in light of the severe damage to concrete containing shale 2068. It is probable that the great difference in surface deterioration between shale 2068 and shales 2066 and 2076 is not entirely due to the greater porosity and absorption of sample 2068, but was at least partially a result of the relative lack of induration of this shale as compared to the others.

Rate of Absorption

The rates of absorption of cherts 2067 and 2077 (Fig. 3) apparently had little influence on freeze-thaw durability of these materials. Material in all three specific gravity groups of chert 2067 attained nearly maximum absorption for the test in only five minutes, while the absorption of chert 2077 for the same specific gravity groups for the first five minutes was only about 25 percent of its total absorption. Although the total porosities and absorptions of these two cherts were similar, the rates of absorption indicate the two materials have considerably different pore systems. Chert 2067 is obviously more permeable than chert 2077. This difference in permeability between cherts 2067 and 2077 is reflected in degrees of saturation obtained by vacuum-saturating these two aggregates (Table 7). At all specific gravity levels chert 2067 had higher saturation coefficients than chert 2077. In spite of this difference in permeability and its resulting difference in degree of saturation, these two cherts resulted in similar deterioration in the freeze-thaw test at all specific gravity levels.

It should be noted that the freeze-thaw tests were conducted under rather severe saturation conditions. The aggregate was vacuum-saturated before mixing the concrete, and the beams were immersed in water for 13 days prior to being subjected to freezing and thawing. They were, of course, reimmersed during each thaw cycle. Under such conditions both cherts maintained a high degree of saturation (Table 7). Under actual service conditions, however, the amount of available water would not always be as great as in these laboratory tests and permeability could have a greater influence on freeze-thaw durability.

The rates of absorption of the shales (Fig. 4) are directly related to the total absorptions of these materials. Those shales with high total absorptions absorbed water rapidly during the first few minutes of the test, following which, water was absorbed at a slowly decreasing rate for the rest of the test. The shales with low total absorptions exhibited a fairly constant increase in absorption throughout the test. As was the case with porosity and total absorption, this greater permeability of certain shales had no influence on the resistance of the shales to deep-seated deterioration. It probably is a factor in surface deterioration, however, since shale 2068 which caused the most popout damage also had the fastest rate of absorption.

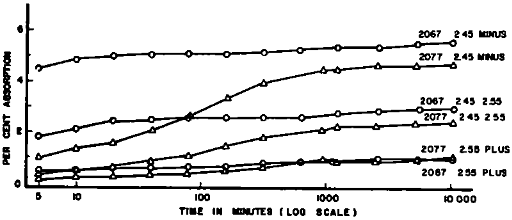

Figure 3. Rates of absorption for different specific gravity fractions of cherts 2067 and 2077.

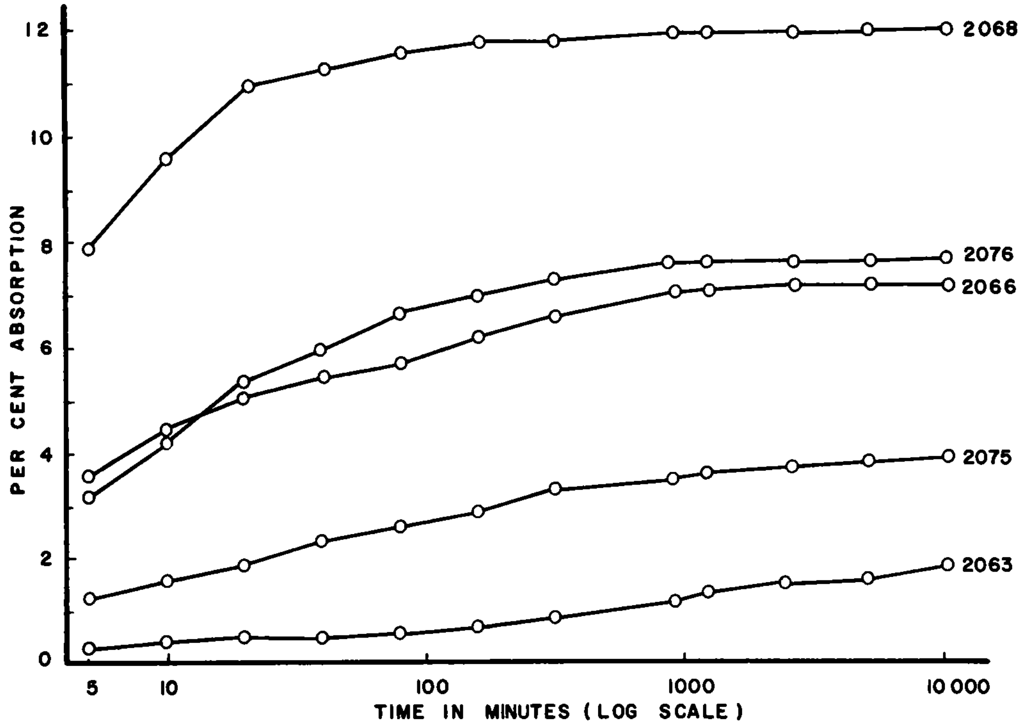

Figure 4. Rates of absorption for shale samples.

MINERALOGY, TEXTURE AND MICROSTRUCTURE

The properties of mineralogy, texture, and microstructure have been grouped together because they were all included in studies utilizing the petrographic microscope. Microscopic petrography has long been a valuable tool in study of the characteristics of rocks. Runner (19) was one of the first to apply petrography to the study of deleterious substances in aggregates. His petrographic investigations were followed by reports of similar studies and description of techniques by Mielenz (20, 21), Rhoades and Mielenz (22, 23), and Mather and Mather (24). In the present study an attempt was made to find a relationship between the results of petrographic studies of deleterious materials and the laboratory freeze-thaw durability of these materials.

Petrographic study was carried out using a Leitz Ortholux petrographic microscope with binocular attachments. Thin sections were made from each of the five shale samples and six chert samples. These sections were studied under transmitted light at magnifications of approximately 100 X to 400X. Since the complete mineral composition of shales is not easily determined by microscopic analysis, X-ray diffraction and differential thermal analysis supplemented microscopic petrography in study of shale mineralogy.

Petrographic analysis of thin sections showed the cherts to be of generally similar mineralogical character. They are composed primarily of microcrystalline quarts and radial chalcedony. Small amounts of coarser-grained secondary quartz, some calcite, and limonite and carbonate rhombs are also present.

Each of the cherts consists primarily of microcrystalline aggregates of quartz grains usually less than 0.01 mm in diameter. The secondary quartz occurs as granular masses which apparently replaced carbonate minerals. The individual quartz grains in these secondary masses range in size from less than 0.01 mm to as large as 0.2 mm. Radiating chalcedony in the chert samples occurs as spherulites, often as much as 0.25 mm wide.

The carbonate and limonite rhombs range in size from less than 0.01 mm to as much as 0.1 mm. Although most of the rhombs consisted of carbonate or limonite, some appeared to have a translucent carbonate mineral in the center surrounded by a rim of opaque limonite. The carbonate rhombs probably formed by replacement of crystalline quartz in the original chert (25), and in turn these rhombs are being replaced by limonite. Limonite also occurs as finely disseminated masses scattered throughout the chert.

One chief mineralogical difference was noted among the chert gravels. Cherts from the southern part of Indiana (especially material from the Ohio River) contain considerably more limonite than those from the northern part of the state. This difference in limonite content apparently had no influence on freeze-thaw durability, however, since all six cherts reacted similarly to the freeze-thaw tests.

It is also of interest to note that little difference in mineralogy was found among the different specific gravity groups for each chert source in spite of the fact that the materials in the different specific gravity groups had considerably different freeze-thaw durabilities. Apparently there is little direct relationship between mineralogy and freeze-thaw durability of Indiana cherts.

An important microstructural feature in chert samples from all six sources is the numerous voids observed in thin sections from material in the 2.45 minus specific gravity range (Figs. 5 and 6). No voids were noted in the 2.45-2.55 and the 2.55 plus ranges. The voids are all fairly large, since voids less than about 30 microns in

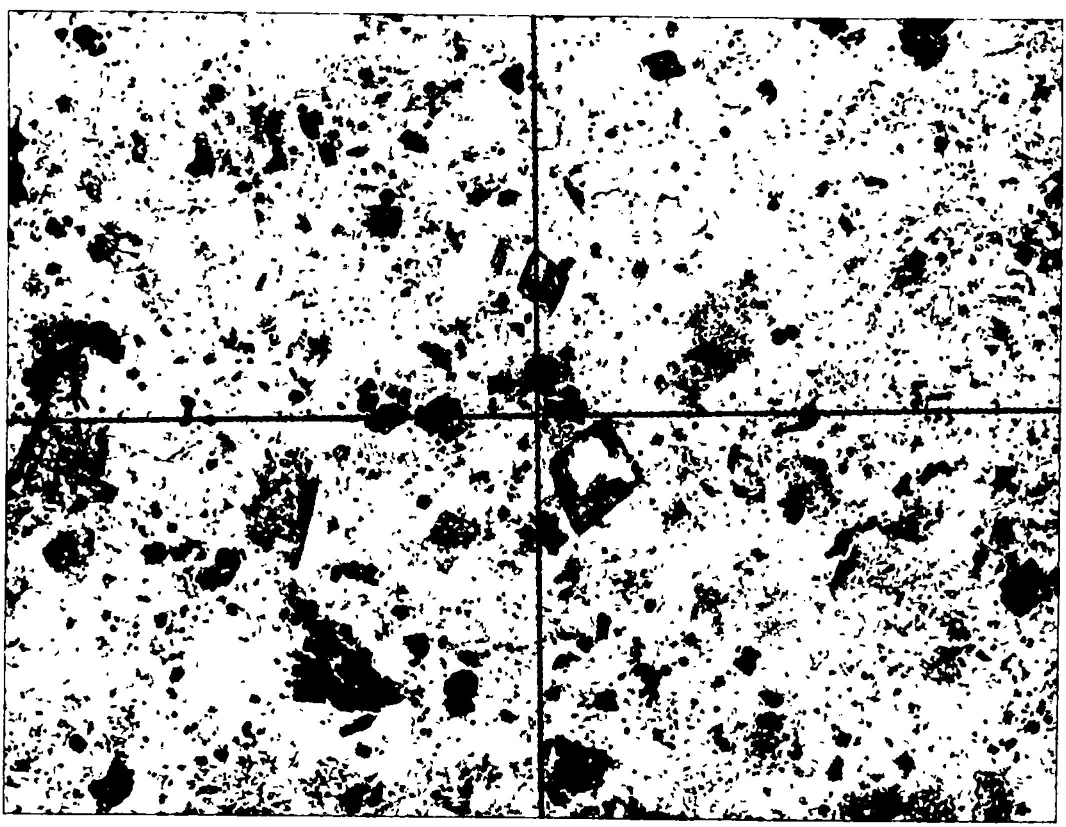

Figure 5. Chert 2077 (s.g. 2.45 minus) in plain light. Limonite (opaque) and carbonate (translucent rhombs in fine-grained quartz matrix. Note void in process of formation by solution of carbonate from large rhomb near intersection of cross-hairs.

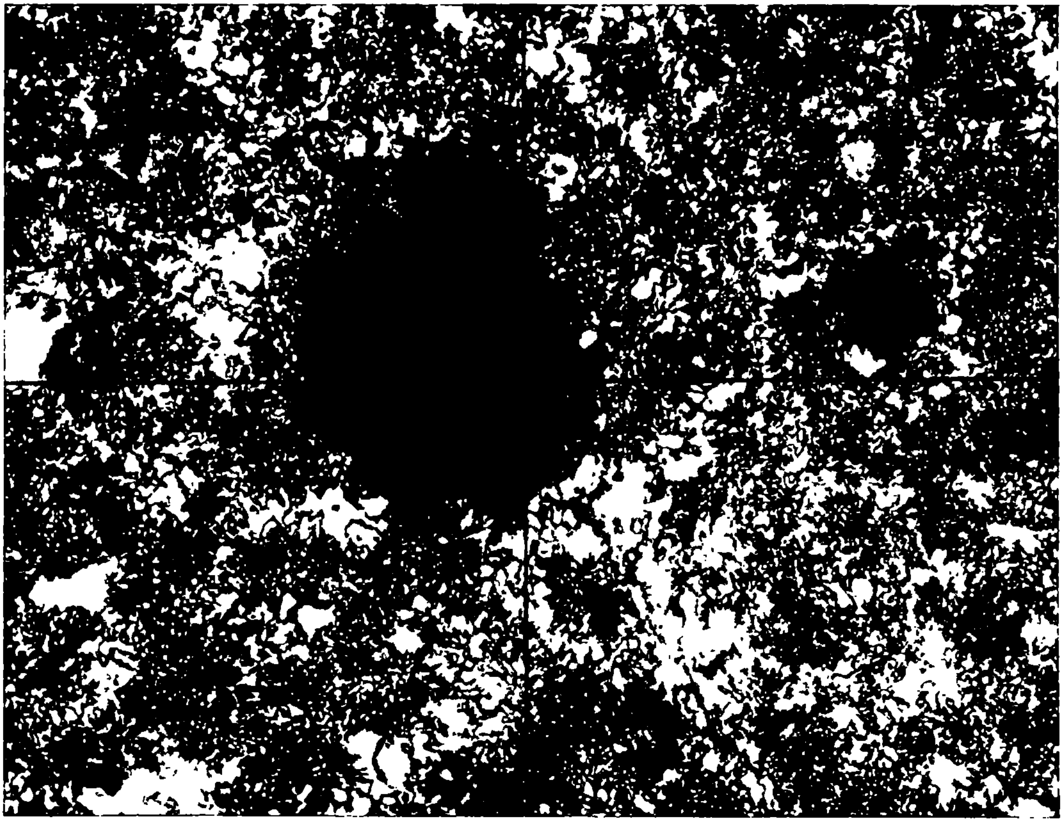

Figure 6. Chert 2063 (s.g. 2.45 minus) between crossed nicols. Large voids surrounded by radial chalcedony and fine-grained quartz matrix.

diameter cannot be detected easily in thin-section study. They range in size up to 0.4-0.5 mm in diameter, but most are less than 0.1 mm in diameter. As noted in the previous section on porosity, the concentration of these voids in the chert with specific gravity less than 2.45 resulted in the relatively high porosity of this lightweight chert, and the high degree of saturation achieved in the lightweight chert fractions is probably also related to the presence of these voids. Voids of this size had previously been recognized in thin sections of lightweight cherts from other states by Wuerpel and Rexford (14) who noted that these voids were related to the lack of durability of the cherts.

The 2.45-2.55 and 2.55 plus specific gravity groups contained practically no voids large enough to be recognized in thin section. Since lack of freeze-thaw durability was found only in the 2.45 minus chert, there is a direct correlation between the presence of these voids and the lack of durability of the lightweight chert. Although this contradicts the theories of Blanks (18) and others, that freeze-thaw deterioration occurs primarily in voids less than 5 microns in diameter, there is a strong possibility (as demonstrated previously in the section on porosity) that the larger voids are prime factors in the freeze-thaw breakdown of lightweight cherts due to the higher degree of saturation afforded the chert by the larger voids.

Other textural properties such as grain size, and presence of rhombic-shaped grains and replaced fossils, apparently had no influence on the freeze-thaw durability of the chert. These characteristics are similar in cherts of all three specific gravity ranges.

Petrographic, X-ray, and differential thermal analyses of the shales indicate a similarity in their general mineralogic composition, but there is considerable variation in certain characteristics. All the shales consist of detrital mineral grains, primarily quartz, in a fine-grained matrix of clay minerals or hydromicas. The chief differences shown by the shales are the relative size and abundance of the detrital minerals and the relative amounts of clay minerals and organic material in the samples.

In order to satisfactorily describe the shales and to point out differences in their petrographic characteristics, and yet avoid repetition, a brief petrographic description of a shale with "average" characteristics will be given, and the mineralogies, textures, and microstructures of the strongest, least porous shale (2063) and the weakest, most porous shale (2068) will be compared with those of the average sample.

The average shale is composed primarily of a fine-grained illite matrix enclosing detrital quartz grains up to 0.04 mm in diameter (0.01-0.02 mm average). It contains considerable organic matter and disseminated limonite and a small amount of chlorite. Loss on ignition for this shale is approximately 12 percent. This description fits shales 2066, 2075, and 2076 fairly well.

Shale 2063, the most indurated and least porous of the shales, contained more detrital quartz and less clay mineral than the average shale. Besides being more abundant, the detrital quartz grains are larger than in the other shales, ranging in size up to 0.07 mm in diameter (0.02-0.03 mm average). The abundance and size of the detrital quartz is sufficient to classify sample 2063 as a silty shale or possibly even a siltstone. This sample also contained more organic material than the other shales as was shown by the 16.7 percent loss on ignition, the highest of all the shales.

Shale 2068, the softest and most porous of the shales, contains more clay mineral and less detrital quartz than the other shales. The size of the detrital quartz grains is about the same as that of the average shale, but the lower percentage of these grains means that the average grain size of shale 2068 is considerably smaller than that of the average shale. The relatively high percentage of illite accounts for the high porosity of this shale, and the combination of high clay and low quartz content accounts for its lack of induration.

Although the differences in relative percentages of clay minerals and quartz apparently have no effect on the tendency of the shales to resist deep-seated deterioration, these mineralogic differences, which influence the textures and microstructures, apparently do affect the amount of surface deterioration caused by the shales. This is especially true for shale 2068. Its high clay mineral content renders it weaker and more porous than the other shales, and thus it is more susceptible to freeze-thaw deterioration.

SUMMARY OF RESULTS

The following is a brief recapitulation of major findings of the study:

1. Freezing and thawing tests of concrete beams containing chert indicated the following:

(a) The source of chert had no effect on freeze-thaw durability.

(b) The only combination of variables resulting in severe deep-seated deterioration was 10 percent of 2.45 minus chert. This combination resulted in deep-seated failure of all beams containing chert from each of the six sources. In addition, 6 percent of 2.45 minus chert caused moderate deep-seated damage in a few cases.

2. Durability factors for the shale beams indicated that no deep-seated deterioration occurred in beams containing 2 to 10 percent of any of the five shales studied. The data included no extremely low durability factors as were found for beams containing 10 percent of 2.45 minus specific gravity chert. Only a few had durability factors below 90, and these few values were seemingly randomly distributed throughout the data.

3. Study of surface deterioration of concrete beams containing chert showed that freezing and thawing caused significant popout damage in beams containing 2.45 minus specific gravity chert. Few popouts were caused by chert having specific gravities of 2.45-2.55 and 2.55 plus.

4. The greatest amount of surface deterioration of the beams containing shale was caused by shale 2068, the most porous and most absorbent of the shales. Shale 2068

caused considerable popout and pitting damage at all four percentage levels, but, as would be expected, the amount of deterioration increased with increasing percentage of shale. The other four shales tested caused no surface deterioration when included in concrete in amounts up to and including 4 percent. At the 6 and 10 percent levels, shales 2066 and 2076, which were more porous and absorbent than shales 2063 and 2075, caused slightly more surface deterioration than the latter.

5. The study of air voids in concrete by means of the linear traverse technique demonstrated that machine-mixed concrete beams with high durability factors had air-void spacing factors lower than 0.01 in., and hand-mixed beams with low durability factors had spacing factors higher than 0.01 in. These results support Powers theory that concretes with spacing factors lower than 0.01 in. are well protected from freezing and thawing deterioration, while those with spacing factors greater than 0.01 in. are poorly protected.

6. Study of the size distributions of pores for cherts 2067 and 2077 indicated a marked increase in percentage of total voids volume consisting of pores larger than 5 microns in diameter with decreasing bulk specific gravity of the chert. For example, in the case of chert 2067, the voids larger than 5 microns in diameter constituted only 20 percent of the total pore space in the 2.55 plus chert, 25 percent in the 2.45-2.55 material, and 50 percent in the 2.45 minus range. Conversely, the voids less than 5 microns in diameter constituted a decreasing percentage of total voids volume with increase in total porosity and resulting decrease in bulk specific gravity.

7. Although the total porosities and absorptions of cherts 2067 and 2077 were similar, their rates of absorption indicate that these two cherts have considerably different pore systems. Chert 2067 was more permeable than chert 2077. Material in all three specific gravity groups of chert 2067 attained nearly maximum absorption after only five minutes of immersion, while the absorption of the same specific gravity groups of chert 2077 for the first five minutes was only about 25 percent of its total absorption.

The rates of absorption of the shales were directly related to the total absorptions. Those shales with high total absorptions absorbed water rapidly during the first few minutes of immersion, following which water was absorbed at a slowly decreasing rate for the rest of the test. The shales with low total absorptions exhibited a fairly constant increase in absorption throughout the test.

8. Petrographic analysis of thin sections showed the cherts to be of generally similar mineralogical character. They were composed primarily of microcrystalline quartz and radial chalcedony. Small amounts of coarse-grained secondary quartz, some calcite, and limonite were also present. One chief mineralogical difference was noted. Cherts from the southern part of Indiana (especially from the Ohio River) contained more limonite than those from the northern part of the state. This limonite occurred both as rhombs and in amorphous form. No differences in mineralogy were noted among the three specific gravity groups for the chert samples.

The shales also presented a similarity in their general mineralogic compositions, but showed considerable variability in certain characteristics. All the shales consisted of detrital mineral grains, primarily quartz, in a fine-grained matrix of clay minerals or hydromicas. The chief differences shown by the shales were the relative size and abundance of the detrital mineral grains and the relative amounts of clay minerals and organic material in the samples.

9. The textures and microstructures of the cherts were all similar. Each chert consisted primarily of microcrystalline aggregates of quartz grains usually less than 0.01 mm in diameter with granular masses of secondary quartz, radiating masses of chalcedony, and carbonate and limonite rhombs. The only notable structural difference in the cherts was that the 2.45 minus fraction of each sample contained numerous voids large enough to be identified in thin sections between crossed nicols. These voids, which averaged less than 0.01 mm in size, but ranged in size up to 0.4-0.5 mm, did not occur in the 2.55 plus and 2.45-2.55 specific gravity groups.

The textures and microstructures of the shales varied considerably. Although all the shales consisted of a fine-grained matrix enclosing detrital quartz grains, the relative amounts of these materials and the sizes of the detrital particles varied enough to influence strongly the strength and hardness of the different shales. All the shales showed preferred orientation of grains.

CONCLUSIONS

Since this study was restricted to certain Indiana cherts and shales subjected to specific methods of test, the conclusions can logically be applied only to similar cherts and shales under similar conditions. However, in some cases, field behavior of the cherts and shales may be inferred from the following conclusions:

1. For a wide variety of cherts, the source of the chert has no effect on its freeze-thaw durability in concrete. Chert does, however, exhibit a definite relationship between its bulk specific gravity and durability in concrete exposed to freezing and thawing. The use of chert having a bulk specific gravity of less than 2.45 (saturated surface-dry basis) in concrete exposed to freezing and thawing should be avoided.
2. 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 (1). Instead chert durability is apparently based on a more complicated interrelationship between total porosity, size of pores, absorption, and degree of saturation. Pores larger than the 5-micron size specified by Sweet permit easier passage of water into immersed aggregates, result in relatively high degrees of saturation, and contribute to freeze-thaw deterioration of lightweight chert. Microscopic studies of polished sections show that these larger pores make up about half the void volume in 2.45 minus specific gravity chert.
3. The petrographic characteristics of the cherts influence the freeze-thaw durability of these materials only in the relationship of these characteristics to porosity of the cherts. For example, although mineralogy of the cherts has no direct effect on their freeze-thaw durability, the presence of carbonate rhombs, which have weathered out to form voids, has lessened the durability of some chert particles.
4. Many shales will not cause deep-seated deterioration of air-entrained concrete beams subjected to laboratory freezing and thawing when included in these beams in amounts up to 10 percent. The inherent structural weakness of these materials may account for this.
5. Different shales cause considerably different degrees of surface deterioration of air-entrained concrete exposed to freezing and thawing. Some shales cause considerable popout damage when included in concrete in amounts as low as 2 percent of the coarse aggregate. Other shales cause little damage when used in amounts up to 10 percent.
6. The durability of the shales studied apparently is related primarily to the porosities and absorptions of these materials; the most porous and most absorbent causing the greatest amount of surface deterioration of concrete in which these materials are used. However, the strength and induration of the shales, as determined by relative amounts of clay minerals and detrital quartz present, also influence the ability of these materials to cause surface deterioration, the softer, weaker materials being less resistant than the harder, stronger ones.
7. As theorized by Powers (10), concretes with air-void spacing factors lower than 0.01 in. are well-protected from freezing and thawing deterioration, while those with spacing factors greater than 0.01 in. are poorly protected.

REFERENCES

1. Sweet, H.S., "Research on Concrete Durability as Affected by Coarse Aggregates." Proc., ASTM, 48:988-1016 (1948).
2. Heinrich, E.W., "Microscopic Petrography." McGraw-Hill Book Company, Inc., New York (1956).
3. Sweet, H.S., and Woods, K.B., "A Study of Chert as a Deleterious Constituent in Aggregates." Engineering Bull. of Purdue University, Research Series 86, 26:5 (Sept. 1942).

4. Wray, F.N., and Lichtefeld, H., "The Influence of Test Methods on Moisture Absorption and Resistance of Coarse Aggregate to Freezing and Thawing." Proc., ASTM, 40:1007-1020 (1940).
5. Thomas, W.N., "Experiments on the Freezing of Certain Building Materials." Building Research Technical Paper No. 17, Department of Scientific and Industrial Research, England (1938).
6. Klieger, P., "Effect of Entrained Air on Strength and Durability of Concrete Made with Various Maximum Sizes of Aggregate." Proc., HRB, 31:177-201 (1952).
7. Walker, R.D., and McLaughlin, J.F., "Effect of Heavy Media Separation on Durability of Concrete Made with Indiana Gravels." HRB Bull., 143:14-26 (1956).
8. Lang, F.C., "Summary of Tests on Effect of Shale in Gravel on Compressive Strength of Concrete." Proc., American Concrete Institute, 23:592-604 (1927).
9. Fears, F.K., "Correlation Between Concrete Durability and Air-Void Characteristics." HRB Bull., 196:17-28 (1958).
10. Powers, T.C., "The Air Requirement of Frost-Resistant Concrete." Proc., HRB, 29:184-202 (1949).
11. Sweet, H.S., "Chert as a Deleterious Constituent in Indiana Aggregates." Proc., HRB, 20:599-620 (1940).
12. Fears, F.K., "Determination of Pore Size of Four Indiana Limestones." Thesis, submitted in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering, Purdue University (1950).
13. Cantrill, C., and Campbell, L., "Selection of Aggregates for Concrete Pavement Based on Service Records." Proc., ASTM, 39:937-945, Discussion, pp. 946-949 (1939).
14. Wuerpel, C.E., and Rexford, E.P., "The Soundness of Chert as Measured by Bulk Specific Gravity and Absorption." Proc., ASTM, 40:1021-1043, Discussion, pp. 1044-1054 (1940).
15. Powers, T.C., "Basic Considerations Pertaining to Freezing-and-Thawing Tests." Proc., ASTM, 55:1132-1155 (1955).
16. Axon, E.O., Willis, T.F. and Reagel, F.V., "Effect of Air-Entrapping Portland Cement on the Resistance to Freezing and Thawing of Concrete Containing Inferior Coarse Aggregate." Proc., ASTM, 43:981-994 (1943).
17. Lewis, D.W., and Dolch, W.L., "Porosity and Absorption." Spec. Tech. Publ., ASTM, No. 169, Significance of Tests and Properties of Concrete and Concrete Aggregates, pp. 303-313, 1956.
18. Blanks, R.F., "Modern Concepts Applied to Concrete Aggregates." Proc., ASCE, 75:441-468 (1949).
19. Runner, D.G., "The Value of Petrography in Determining the Quality of Rocks." Public Roads, 18:69, 74, 77 (1937).
20. Mielenz, R.C., "Petrographic Examination of Concrete Aggregates." Bull., Geological Society of America, 57:309-318 (1946).
21. Mielenz, R.C., "Petrographic Examination." Spec. Tech. Publ., ASTM, No. 169, Significance of Tests and Properties of Concrete and Concrete Aggregates, pp. 253-273 (1956).
22. Rhoads, R., and Mielenz, R.C., "Petrography of Concrete Aggregate." Proc., American Concrete Institute, 42:581-600 (1946).
23. Rhoads, R., and Mielenz, R.C., "Petrography and Mineralogic Characteristics of Aggregates." Spec. Tech. Publ., ASTM, No. 83, Symposium on Mineral Aggregates, pp. 20-48 (1948).
24. Mather, K., and Mather, B., "Method of Petrographic Examination of Aggregates for Concrete." Proc., ASTM, 50:1288-1313 (1950).
25. Walker, T.R., "Carbonate Replacement of Detrital Crystalline Silicate Minerals as a Source of Authigenic Silica in Sedimentary Rocks." Bull., Geological Society of America, 71:2, 145-152 (Feb. 1960).

Discussion

F. E. LEGG, JR., Associate Professor of Construction Materials, University of Michigan, and Materials Consultant, Michigan State Highway Department—The authors of this study are to be congratulated on making such an intensive effort to determine the behavior of chert and shale in concrete when exposed to freezing and thawing and, even more important, to discover the fundamental properties of these rock types which caused their deleterious action.

The remarkable increase in commercial heavy media beneficiation in recent years now gives opportunity to study the behavior of the heavier gravel constituents under actual service conditions thus permitting comment on the validity of the authors' laboratory findings. The low specific gravity types are quite effectively discarded by the media process and deleterious action in concrete made with the beneficiated gravel is presumably confined to the higher gravity particles. Observations of concrete in service containing such gravel led to the conclusion that even the high gravity cherts will cause popouts although the action may be delayed a few years.

Brief survey was recently made of chert popouts in two air-entrained concretes made with beneficiated gravel from the same southern Michigan source—a pavement 2 yr old and the top deck of an open parking structure 4 yr old. Where they could be found, the fragments of chert remaining in the bottom of the popouts were removed and their saturated surface dry gravities determined in heavy liquids. Table 10 gives the gravity range of the chert extracted from the bottom of each popout. It is observed that the majority of chert popouts in the younger pavement are in the range of 2.40 to 2.50 specific gravity whereas in the older parking structure they are in the range of 2.50 to 2.60. In both there are a substantial number having gravities above 2.45 which is contrary to that which might be anticipated from the laboratory findings of the present authors whose second conclusion is, in part, "Apparently only chert with a bulk specific gravity of less than 2.45 (saturated surface dry basis) will cause either deep-seated or surface deterioration of air-entrained concrete in which it is used."

Laboratory data leading to a prediction of deleterious action from even the heavier cherts were presented to the Highway Research Board in 1956 (1). Gravel having an average durability factor of 87 was diluted with 10 percent chert of four specific gravity ranges—minus 2.45, 2.45 to 2.50, 2.50-2.55 and 2.55 plus. Six test beams, two each from three batches of air-entrained concrete, were freeze-thaw tested using the rapid air method, ASTM C-291. Figure 7 shows the results of these tests, with the 95 percent confidence interval designated by shaded areas. Where overlapping of confidence limits occurs, it indicates that chance plays such a part that discrimination between the concrete may not be justified. The data on this basis indicate superior performance for the undiluted gravel over that diluted with 10 percent chert having a specific gravity of 2.55 or lower. Also, the undiluted gravel or that containing 10 percent chert of 2.55 plus gravity exhibits superior performance to that containing chert of 2.50 gravity or lower. The data thus indicate a scale of durability of chert—with diminishing durability as the gravity goes down, rather than the abrupt change exhibited in the Purdue studies.

The disparity in results between the Indiana and Michigan studies indicates the likelihood of subtle differences in conduct of freeze-thaw tests, the influence of which are not now well understood. The possibility of actual differences between the cherts from the two areas cannot be positively ruled out, but in view of the recent report of Cook (2) and the HRB Report of Cooperative Freezing-and-Thawing Tests of Concrete (3), it seems more likely that minor differences in technique may be the reason for lack of concordance.

TABLE 10

SPECIFIC GRAVITY OF CHERT FRAGMENTS EXTRACTED FROM POPOUTS

Bulk Specific Gravity, Saturated Surface Dry	Number of Chert Popouts	
	Pavement 2 Yr Old	Parking Structure 4 Yr Old
2 30 - 2 35	1	-
2 35 - 2 40	0	-
2 40 - 2 45	2	1
2 45 - 2 50	4	1
2 50 - 2 55	1	6
2 55 - 2 60	-	5
2 60 - 2 65	-	1
Total	8	14

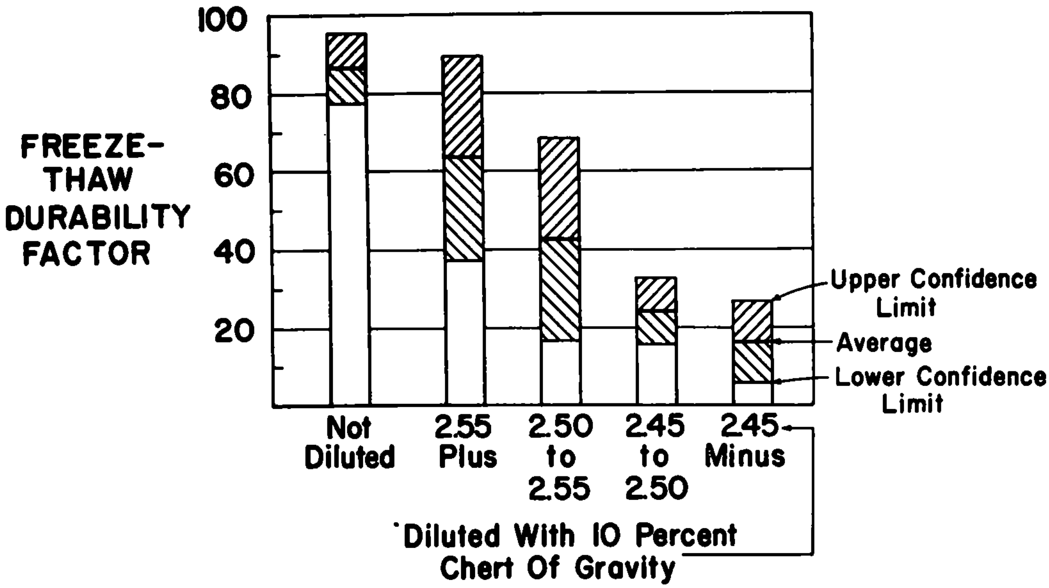

Figure 7. Freeze-thaw durability factors of a Michigan gravel diluted with 10 percent chert of different specific gravities.

Briefly, caution is suggested with respect to the authors' observations regarding lack of popouts of chert having a gravity greater than 2.45. Severity of exposure, or differing quality of chert, apparently influence chert behavior in exposed concrete. The writer's experience leads to the conclusion that much heavier cherts will actually cause popouts, but it is agreed that the lighter cherts are more active.

REFERENCES

1. Legg, F.E., Jr., "Freeze-Thaw Durability of Michigan Concrete Coarse Aggregates." HRB Bull. 143 (1956).
2. Cook, H.K., "Automatic Equipment for Rapid Freezing and Thawing in Water." HRB Bull. 259 (Jan. 1960).
3. Report on "Cooperative Freezing-and-Thawing Tests of Concrete." HRB Sp. Rept. 47 (1959).

J.F. McLAUGHLIN and R.L. SCHUSTER, Closure—The authors agree that the properties of chert giving rise to deterioration, either popouts or deep-seated failure, do not change dramatically at any given specific gravity level. Studies have shown, however, that when this chert is in concrete wherein the remainder of the aggregate is uniformly of excellent quality, the effect of these properties on the concrete is practically nil unless the chert has a gravity of 2.45 or less. Professor Legg justifiably points out that this may not be the case when the chert is in more heterogeneous surroundings. The authors were attempting in this study to quantify the effects of chert on an otherwise uniform, highly durable concrete and conclusions, as stated in the paper, must necessarily be limited by these conditions.

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