Identification of Coarse Aggregates That Undergo Destructive Volume Changes When Frozen in Concrete

RICHARD D. WALKER
Associate Professor of Civil Engineering, Virginia Polytechnic Institute, Blacksburg, Virginia

ABRIDGMENT *

•THE OBJECTIVE of this project was to develop a quick test method to distinguish deleterious particles in aggregate and predict their behavior in concrete subjected to freezing and thawing.

Thirty-two different concretes were used to fabricate a total of 281 3- by 3- by 16-in. specimens. Eight coarse aggregates were used ranging from trap rock and limestone crushed stones to a variety of gravels from glacial and nonglacial sources. Concretes were made from the whole coarse aggregates as well as from certain of their individual constituents.

Most of the concrete specimens were exposed to alternate cycles of freezing and thawing while in water in accordance with ASTM test designation C 290-61T. The specimens were measured for length, weight, and dynamic modulus at the end of specified numbers of cycles. Also, length change measurements were made with a Whittemore strain gage at approximately 2-deg intervals during most of the initial freezing-and-thawing cycle as well as during several other cycles. The freezing rate was approximately 20 F/hr.

Companion specimens, in some cases, were also exposed to a single slow freeze cycle in a deep freeze unit. Length change and temperature measurements were made at 10- to 15- min intervals over a 4-hr period. The freezing rate of this procedure was between 5 and 10 F/hr.

From the data obtained, correlations were made between concrete durability as measured by dynamic modulus and certain characteristics of the temperature vs length change curve.

AGGREGATES STUDIED

The eight coarse aggregates used are briefly described in the following.

A. A high quality quartzite gravel from the eastern Piedmont region. This is a hard durable aggregate with an excellent field performance record. It was used both for comparative purposes and for blending with other aggregates.

B. A high quality trap rock crushed stone from the eastern United States.

C. A crushed limestone having unknown field performance from southeastern Canada.

D. A gravel of very heterogeneous composition from a river terrace of the glaciated midwest. This aggregate has a fair field performance record.

E. A blast furnace slag from the eastern United States.

F. A midwestern river gravel, composed mostly of chert, having a poor field performance record.

G. A chert gravel from the Ohio River having an extremely poor field performance record.

*This is an abridgment of a paper presented at the 44th Annual Meeting. The complete paper as then presented is available from the HRB at cost of reproduction and handling. However, an interim report of this project (NCHRP 4-3(1) FY '63) has been published as NCHRP Report 12.

Paper sponsored by Committee on Mineral Aggregates.

H. A reject midwestern glacial gravel which floated in a commercial heavy media plant operating at a media gravity of 2.55. This aggregate has no field performance record because it is not used in pavement structures.

In addition to these eight aggregates, individual rock types were separated by hand from aggregates D and H in sufficient quantity to be used in concrete. Aggregates D and H were the most heterogeneous gravels obtained and the only ones from which sufficient quantities of many different rock types could be obtained. The other six aggregate sources were of more or less uniform composition. The individual rock types separated were as follows: (1) high calcium limestone, (2) chert, (3) limey chert, (4) calcitic siltstone and sandstone, (5) quartzite, (6) non-end members of limestone and dolomite, (7) dolomite, (8) weathered rounded trap, (9) shale, and (10) granite.

Concrete Mix Design

All concrete mixes were designed using a blend of three brands of low alkali type I cement, a cement factor of 5.5 sk/cu yd, and air content of 5.5 percent, and sufficient water to obtain a 3-in. slump. A single high-quality quartzite sand was used throughout the entire project. All course aggregates were vacuum saturated and all specimens were cured in lime water for 13 days.

Results and Discussion

Two terms must be defined before the results of the study can be presented and discussed:

1. The minimum 5 F temperature slope, b_1, is the minimum slope that can be found, within a 5 F or more range, on the length change vs temperature curve obtained during the first freeze of each specimen. It is in units of 10^{-4} in./deg F measured over a 10-in. gage length.
2. The time slope, b_t, is the minimum slope that can be found within a 20 min (⅓ hr) or greater range on the length change vs time curve for the slow freeze cycle specimens only. The units of b_t are 10^{-4} in./hr (measured over a 10-in. gage length).

Table 1, herein (Table 3 of original paper; Table 5, NCHRP Report 12), gives the relationship between DF_{100} and b_1 for all specimens that underwent the alternate freezing and thawing cycle procedure. The values are the averages of all specimens for the particular mix design. Although a certain amount of variability is evident, a fairly good separation is obtained between groups I and II and groups II and III.

Figure 1, herein (Fig. 3 of original paper; Fig. 6, NCHRP Report 12), shows the relationship between DF_{100} and b_1 for the concretes made with the individual particle types. With the exception of aggregate U, a shale, a fairly good relationship is shown.

Figure 2, herein (Fig. 8 original paper; Fig. 11, NCHRP Report 12), shows the relationship between DF_{100} and b_t for the specimens frozen in the deep freeze. Omitting points F-2 and L for purposes of illustration, an extremely high correlation coefficient is obtained. If b_t were a good prediction of the quality or durability of concrete exposed to freezing and thawing, specimens could be tested using only a deep freeze unit, a Whittemore strain gage, and an instrument for keeping time. Expensive temperature measuring equipment would thus not be required.

On the basis of these results, it is felt that one or both of the test procedures described has excellent potential as at least a preliminary test to separate potentially bad aggregates from good aggregates. In no case did a mix design have a b_1 or a b_t value of less than zero, indicating that dilation never took place, with aggregates falling in the good category. Perhaps if a b_1 or b_t value of less than zero were obtained, it might be decided that more lengthy tests might be performed. All of the traditional freezing-and-thawing tests and the one cycle freeze test described by Powers (1) are time-consuming. It is hoped that future testing will show that a test of the nature described in this study will prove practical.

REFERENCE

1. Powers, T. C. Basic Considerations Pertaining to Freezing-Thawing Tests. Proc. ASTM, Vol. 55, pp.1132-1155, 1955.

TABLE 1

RELATIONSHIP BETWEEN MINIMUM 5 DEG TEMPERATURE SLOPE, b_1 (10^{-4} IN./DEG F) AND 100 CYCLE DURABILITY FACTOR, ALL MIX DESIGNS

Group I Extremely Bad[a]			Group II Bad[b]			Group III Fair[c]			Group IV Good[d]		
Mix	DF_{100}	b_1[e]	Mix	DF_{100}	b_1[f]	Mix	DF_{100}	b_1[g]	Mix	DF_{100}	b_1[h]
F-1	3	-2.602	F-2	12	+0.075	D	66	+0.102	A	97	+0.882
F-3	3	-0.549	I	25	-0.398	E	59	+0.143	B	98	+0.835
G	1	-5.768	J-1	9	-0.014	J-3	44	+0.489	C	84	+0.316
H	5	-2.072	J-2	14	+0.063	J-6	66	+0.450	K-1	85	+0.029
N	4	-1.316	J-4	12	-0.073	R	51	+0.178	K-2	93	+0.010
W	1	-3.274	J-5	7	-0.220	S	46	+0.101	K-3	94	+0.167
			L	10	-0.300	U	40	+0.642	M	95	+0.291
			O	8	-0.863				Q	95	+0.264
			P	27	-0.332				T	98	+0.698
									V	95	+0.813
Avg.	3	-2.596	Avg.	14	-0.229	Avg.	53	+0.330	Avg.	93	+0.431

[a] DF_{100} = 0-5.
[b] DF_{100} = 6-30.
[c] DF_{100} = 31-80.
[d] DF_{100} = 81-100.
[e] Ninety percent confidence limits: -4.095 to -1.097.
[f] Ninety percent confidence limits: -0.409 to -0.049.
[g] Ninety percent confidence limits: +0.139 to +0.461.
[h] Ninety percent confidence limits: +0.233 to +0.629.

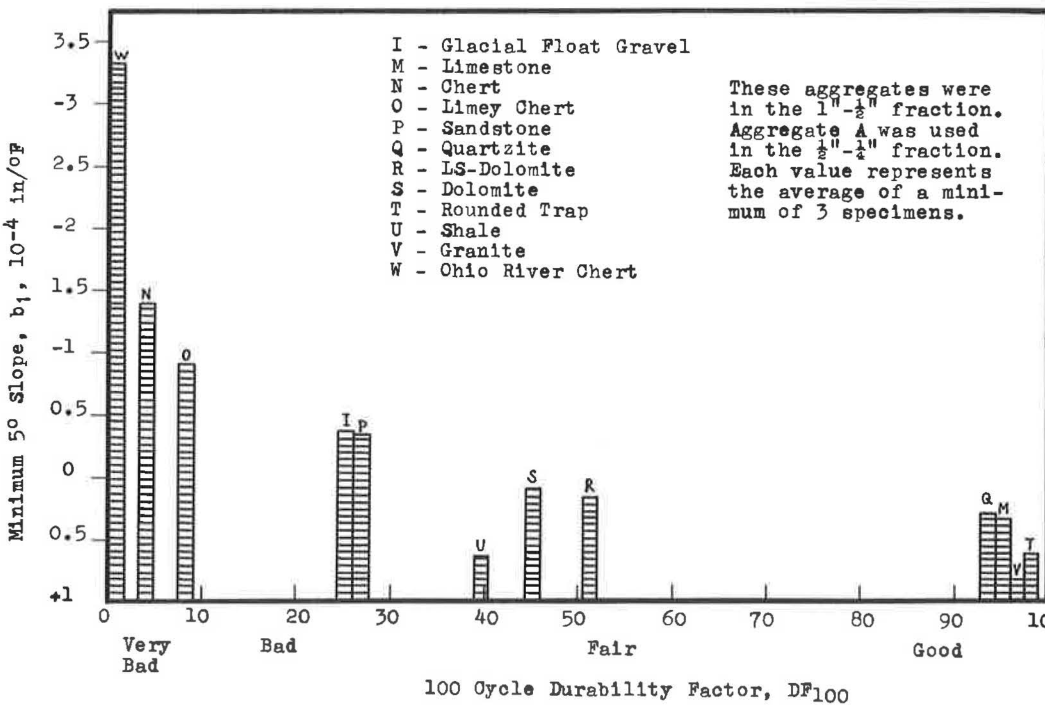

Figure 1. Relationship between DF_{100} and b_1 for individual particle types.

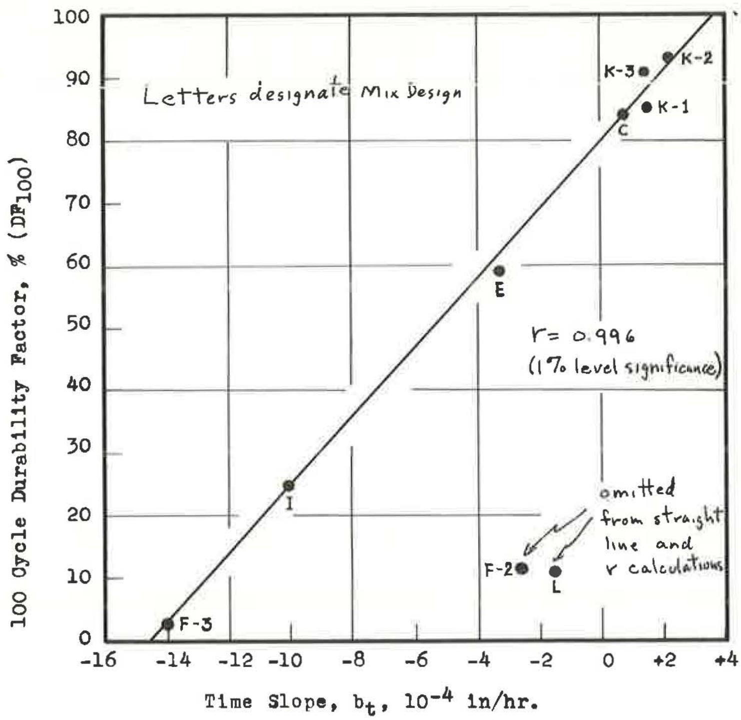

Figure 2. Relationship between DF_{100} and b_t for specimens frozen in deep-freeze.

Discussion

BAILEY TREMPER, Consulting Engineer, Riverside, California—Professor Walker has adopted the term b_1 as a measure of length changes in concrete during the first freeze of each specimen. A negative value of b_1 signifies an increase in length. Conversely, if b_1 is positive a decrease in length is indicated.

The value of b_1 as used by Walker is evidently the sum of changes due to thermal effect and that produced by dilation of the specimen due to the effect of freezing. Since the thermal coefficient indicates shortening during descending temperature, its value is positive. The thermal value should be substracted from the total observed length change to obtain the value of dilation due to the freezing effect alone.

The thermal coefficient of expansion of concrete is generally in the range of 3 to 6 × 10^{-6} which is equivalent to 0.3 to 0.6 × 10^{-4} measured over a 10-in. gage length, which is the unit used by Walker. These are the values observed after moisture in the gel pores and the capillary pores has reached equilibrium following a change in temperature. Helmuth (2) has shown that when hardened cement paste has been subjected to a change in temperature at a rate comparable to that used by Walker, the observed thermal coefficient is approximately double the value found after moisture equilibrium has been established.

An increase in the freezing rate of concrete would be expected to increase the apparent thermal coefficient, and presumably the amount of dilation also, but the relative effect on the two factors would not necessarily be the same. Examples of the effect of freezing rate on b_1 for two concretes are shown in Figures 6 and 7 of original paper (Figs. 9 and 10, NCHRP Report 12). For concrete F_2, b_1 was zero under rapid freezing but increased to +0.19 under slow freezing. For concrete L, b_1 was -0.3 under rapid freezing but increased to +0.2 under slow freezing. Although these results may indicate that slow freezing was less destructive, such a conclusion cannot be confirmed without knowledge of the effect of freezing rate on the apparent thermal coefficient.

Professor Walker did not report the thermal coefficients of the concretes he tested, but since the values of b_1 as indicated in Table 1, herein, were obtained under decreasing temperature rates of from 13 to 26 deg F/hr, the change due to the thermal effect alone could well have been equal to the observed values of b_1 as reported for group IV (good) aggregates. In other words, these concretes may have exhibited nothing but the natural thermal change in length, and therefore would be rated as "good" on the basis that they developed no dilation.

For the poorer concretes of Table 1, herein, (those exhibiting negative values of b_1), the value of dilation would be the value of b_1 minus the value of the thermal coefficient and would be considerably more negative than b_1 (would exhibit greater dilation).

Professor Walker states that specimens falling in the fair to good categories did not evidence overall expansion usually. Therefore, he concluded that b_1 would be a better measure of performance. The only aggregate he tested that was known to have a "fair" performance record was aggregate D. (The remaining aggregates listed in the fair category in Table 1 were so placed solely because of results obtained by ASTM Designation: C 290-61T, Rapid Freezing and Thawing in Water.) Concrete D developed a b_1 value of 0.102×10^{-4} in. per deg F. If any reasonable value of thermal coefficient is subtracted from this value the result would be negative, indicating actual dilation, the value of which it should have been possible to measure. Therefore, Professor Walker's choice of b_1 as a measure of performance does not appear to be superior to that of dilation.

If all of the concretes tested by Professor Walker had the same thermal coefficient and if this value were subtracted from the reported values of b_1, the separation into four groups indicating good, fair, bad and extremely bad in Table 1 would not be changed. But, it is reasonably certain that all concretes did not have the same thermal coefficient. Therefore, a grouping according to dilation would undoubtedly have brought about changes in rating. The ratings during the first freeze then would not necessarily agree with that shown by the 100-cycle durability factor.

This brings up the question of the validity of using the test for rapid freezing and thawing in water as a measure of field performance. Professor Walker indicates that he questions its validity. Powers (1) has questioned the rapid freezing methods because of the high rate of temperature drop and the unrealistically high internal hydraulic pressures that are developed. Other features of the test as conducted which are not in accordance with the principles set forth by Powers are

1. The use of 1-in. maximum size aggregate particles unless this is the largest size that will be used in the work.
2. Testing the concrete in a saturated condition, whereas in the work, most concrete is afforded some opportunity to dry between the time of construction and exposure to freezing conditions.
3. The fact that the test gives no indication of the performance of partially dried concrete after it is exposed to moisture during winter conditions which vary from location to location.

Professor Walker has stated that his objective was to develop a quick, simple and inexpensive method of test to distinguish deleterious particles in aggregate and predict their behavior in concrete subjected to freezing and thawing. It is not difficult to devise such a test that will distinguish between aggregates of unquestionably poor performance and those of unquestionably excellent performance. The problem lies in rating marginal aggregates that may give satisfactory performance under moderate exposure but not under more severe freezing-and-thawing conditions. It is not believed that Professor Walker's approach is promising for the latter situation, because of the considerations outlined previously.

Although low testing expense certainly is to be desired, the fact that substantial additional costs may be involved in transporting large quantities of aggregates tends to overshadow possible savings in testing expense. Usually, automatic recording methods for measuring length changes will reduce the operator's time sufficiently to make such methods less costly than manual measurements.

Finally, it appears that true economy will result only if the length and severity of winter exposure at a particular construction site is evaluated to select the appropriate test procedure for such conditions.

Reference

2. Helmuth, R. A. Dimensional Changes of Hardened Portland Cement Pastes Caused by Temperature Changes. Proc. Highway Research Board, Vol. 40, pp. 315-356, 1961.

R. D. WALKER, Closure—The author greatly appreciates the thorough review given to this paper by Mr. Tremper. In the light of his discussion it is felt that certain points should be emphasized.

The object of this study was to develop a simple, short, and inexpensive test procedure for evaluating aggregates. The data shown in Figure 2, herein, indicate excellent promise of correlation between time slope, b_t, and durability factor. With this procedure, only time and length change are being measured; thus, a correction for thermal coefficient of the concrete would not be practical. With each concrete made of the same constituents except for the coarse aggregate, the correction would be minimal.

Mr. Tremper points out that automatic freeze-thaw test specimens showed lower temperature slope (b_1) values than those frozen in the deep freeze unit. This is certainly due to a lower freezing rate and to the specimen being frozen in air rather than in water. However, placing the specimen in a 0 F environment should provide reasonably standard freezing conditions which are essential in the development of a test such as this.

Concerning severity of test, as long as correlation of results is ultimately based on field conditions, it should not matter, in most cases, how severe the test is if it will permit differentiation of performance. An aggregate such as the slag (aggregate E) used in this study would be an exception, due to its large pore sizes. It had a durability of only 59, yet in Maryland it has an excellent field performance record.

Mr. Tremper commented on the inadvisability of restricting the maximum aggregate size to 1 in. in the tests described. The test could easily be modified to use larger specimens which could accommodate larger aggregate; however, our studies indicate that there is at least some doubt as to the effect of top size of aggregate on results. As for testing the concrete in a saturated condition, the pros and cons could be presented in a lengthy discussion. In this study saturation was used to promote greater standardization of the test as well as to permit quick results.

This study does not provide a solution to all our problems but only a beginning towards a standardized, simple test that will be of value for at least the preliminary separation of the very bad and very good aggregates. It is believed it can accomplish more than that. A standard test cannot hope to duplicate field environment exactly. A test which does attempt to do this, in the author's opinion, will lose much of its practical value.