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This report has been reviewed by a group other than the authors according to procedures approved by a Report Review Committee consisting of members of the National Academy of Sciences, the National Academy of Engineering, and the Institute of Medicine.

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DEVELOPMENTS IN THE PREDICTION OF POTENTIAL STRENGTH OF CONCRETE FROM RESULTS OF EARLY TESTS

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G. W. Steele, Materials Control, Soil and Testing Division, West Virginia Department of Highways

Relationships among degree hours of maturity and compressive strengths of untreated, 1 to 3-day-old concrete cylinders are used to develop equations for predicting minimum potential 28-day strength. The one-sided confidence limit on values is approximately predicted at f_c - 600 psi (f_c - 4140 kPa). From the results of early tests, one equation produces predicted values for mixtures of the known maturity and the known cement factor, one requires only that specimens be tested at a constant maturity, and one requires only that the maturity be known. Equations are derived by statistical methods and a regression analysis. The data were obtained from a designed experiment made up of 200 cylinders, 4 cement factors, and tests at 1, 2, 3, and 28 days. Equations are based on the nonlinear time-temperature, or maturity, concept of the rate of gain of strength first advanced by Plowman. Predicted results are limited to 28 days and have an average coefficient of variation of 12 percent. Also discussed are the findings that predictions based on the compressive strengths of untreated cylinders, cured at about 73 F (22.8 C), are as accurate as those based on cylinders conditioned by a 3½-hour immersion in a 200 F (93.3 C) water bath. It was found that the compressive strengths not exceeding 1,800 psi (12 400 kPa) of early test specimens tested with Celotex pressure pads are nearly equal to and have about the same standard deviation as those tested with sulfur mortar caps.

THIS paper presents findings of a continuation of previously published work (1) to investigate the practicality of using early tests of concrete cylinders for the quality control of concrete. The scope of the study described includes concrete of four cement constants proportioned to simulate field conditions where part of the cement was inadvertently omitted. One to 3-day-old specimen cylinders were tested. Results of tests, at early ages, based on sulfur mortar caps were compared with results based on Celotex pressure pads. The findings indicate that 1 to 3-day-old specimens can be tested without pretreatment or special conditioning to provide reliable estimates of minimum compressive strengths at 28 days.

In addition, the findings indicate that these specimens can be prepared for tests by means of disposable pressure pads in lieu of the usual sulfur mortar caps and at a savings of time and equipment requirements. The data obtained made possible the simplification of the previously developed prediction equation (1, 2) and the development of two alternate prediction equations.

OBJECTIVES

The objectives of the work described were to determine the possibility of further increasing the practicality of the applications of previous research (1, 2) and to develop prediction equations for concrete mixtures not previously investigated.

The specific objectives established for our study are as follows:

1. To determine the feasibility of basing predictions of potential 28-day strength on concrete cylinder specimens made and cured at ambient temperatures of approximately 73 F (23 C). In previous work (1, 2), the majority of the specimens were subjected to conditioning by immersion in hot water or by heating in an oven or in an autoclave for short periods before the test. Testing concrete compressive specimens without pretreatment accomplishes appreciable savings in time and equipment cost. This would be particularly advantageous under field conditions where equipment and personnel for pretreatment may not be available.
2. To determine the feasibility of substituting pressure pads of semicompressible material for the usually specified sulfur mortar caps (AASHO T 231) with which test specimens were prepared in previous work (1, 2). The purpose is to eliminate otherwise required equipment or labor and to reduce the time involved in making early tests.
3. To develop equations for predicting from the results of early tests the potential minimum 28-day strength of concrete made with significantly reduced cement factors. The purpose is to develop prediction equations appropriate for use when early tests indicate abnormally low compressive strengths.

RESEARCH APPROACH

Prediction of Concrete Strength

Since the 1930s, much work has been done to determine whether or not concrete is of satisfactory quality before the results of tests are made known a considerable time after the placement of the concrete in construction. A report by the Ontario Department of Highways (3) lists 87 references relating to methods for use in estimating the later strength of concrete from early test results. Interest in these methods has been intensified by the adoption of the concepts of quality assurance by many segments of the highway construction industry. The availability of early test results, which can be used as a basis for immediate action, is inherent to the optimization of quality assurance systems.

As previously mentioned, much work has been done in this area. Investigations by others (4) have had various objectives and have resulted in procedures and prediction equations appropriate to these objectives. In general, the underlying concept has been that the gain of strength of concrete from the time of final set is a function of a nonlinear time-temperature relationship. Accordingly, previous work has involved acceleration of the gain of strength by some method of increasing the temperature of the concrete specimens after a short waiting period following their fabrication. This was accomplished by immersion in hot-water baths, low-pressure steam curing, oven heating, or autogenous curing (AASHO C 684-71T). In some cases, where the objective was to obtain an early estimate of the 28-day strength of laboratory-mixed design mixtures, the plastic concrete or concrete that had reached the condition of final set was heated; in others the concrete was heated after curing at ambient temperatures. After pretreatment, the specimens were usually tested in compression, and a mathematical formula was used to predict the strength at some later period, usually 28 days. However, at least in one case an attempt was made to predict the ultimate strength of the concrete (5).

A review of the literature indicated that most previous approaches to the problem of relating the strength of concrete at an early age to that at 28 days suffered from several deficiencies. Procedures often required that the early tests be made at fixed intervals after the specimens were formed. Most of these procedures were not very adaptable to conditions associated with highway construction where there are differences in early curing temperatures and variations in the time required to transfer specimens from the job site to the testing facility. The methods of preparing specimens for the

test were sometimes inconvenient or required extensive equipment. In addition, the mathematical equations used for prediction of potential strength had not proved to be entirely reliable, and the degree of confidence that could be placed in the prediction was uncertain. However, in view of the normal variation of results of tests of concrete made and cured under standard conditions and the knowledge that, for quality control purposes, all that is required is a prediction of minimum potential strength with an acceptable degree of confidence, it was believed that a satisfactory method could be developed of evaluating concrete quality at early ages that would be comparable in significance with 28-day tests.

Accordingly, a relatively new quality assurance procedure for determining the early strength of concrete was developed and evaluated by laboratory experiment and a limited amount of field application (1, 2). The evaluation of these investigations indicates that the procedure used and the formula developed for predicting potential strength are reliable over the range of conditions investigated.

In the work previously reported (1, 2), 0 to 72-hour-old specimens were pretreated by heating in a water bath, oven, or autoclave before testing. It was found that the results of tests of specimens conditioned by any of these methods could be used for the purpose of predicting 28-day strengths with acceptable accuracy. However, in one series of tests where the cylinders were tested at 24, 48, and 72 hours without any pretreatment or heating, predictions based on the results were as accurate as, if not more accurate than, those obtained from specimens pretreated by any of the heating methods.

The equation that was fitted to the experimental data was a modification of the Plowman equation and has the form

$$S_M = S_m + b\ (\log M - \log m) \qquad (1)$$

where

S_M = predicted normal compressive strength at maturity M;
S_m = measured compressive strength at maturity m;
M = degree hours of maturity under standard conditions, i.e., when cured at about 73 F (23 C);
m = degree hours (F hours) of maturity of specimen at time of early test after conditioning (hours of age × ambient temperature) + C;
C = degree hours (F hours) of maturity determined by autogenous heating, method of preparation or conditioning, residual temperature at time of test, and possible unknown factors; and
b = slope of prediction line.

Equation 1 plots as a straight line on semilog graph paper and differs from the Plowman equation in that the line of prediction is projected from the actual value of the early test S_m instead of from the intercept, which was given a constant value by Plowman. This increases the accuracy of prediction since both early and 28-day strengths are affected by sample-to-sample variation. Because of this variation, plotted lines representing tests on different samples are not likely to have the same intercept.

Capping of Concrete Test Specimens

The previous work proved the practicality of predicting the potential strength of the normal concrete from the results of early tests with a reasonable degree of assurance when sulfur mortar caps were used. It would appear that the practicality of the test procedure for quality assurance purposes would be increased if it were further simplified by eliminating the capping operation. This would result in a savings of time, equipment, and materials and would be of particular advantage when tests were made

in a field laboratory.

From Hudson's personal communication with Walker in the 1930s, it was learned that some experimentation had been conducted using Celotex pressure pads with some degree of success. A literature search failed to give any specific information on this matter; however, there is some discussion to the effect that sheet materials had been used, that there was a reduction in strength, and that the reduction was less when low-strength cylinders were tested. Limited experimentation indicates that concrete with 1,900 to 3,700-psi (13 000 to 26 000-kPa) observed strengths will have observed strengths reduced by approximately 8 percent when tested with corrugated cardboard pads in lieu of standard sulfur mortar caps and that the coefficient of variation is less than 10 percent. A reduction in observed strength of this magnitude would not be unacceptable in this particular application since the objective is not necessarily to obtain a precise measure of strength but a value from which future minimum strengths can be predicted.

Prediction Equation 1

Although equation 1 developed in the course of the previously reported work (1, 2) has been shown to produce reasonably accurate estimates of potential 28-day compressive strengths, under normal conditions, there is a theoretical disadvantage. The increased accuracy of prediction of this equation stems from the use of the early test results as the start of the line of prediction. The size of the right side of the equation, representing the estimate of the increased strength due to additional maturity, depends in part on the cement constant of the concrete. Therefore, concrete that was seriously deficient in cement content (evidenced by a very low early strength) could be shown by the equation to have a satisfactory 28-day predicted minimum strength since the rate of gain of strength with increased maturity would be based on normal cement content. For this reason, one of the objectives of this series of tests was to obtain data suitable for deriving a prediction equation that would be applicable to concrete mixtures deficient in cement.

Designed Experiments

The experiment designed to accomplish the objectives of the research reported here was designated as series V. The primary purpose of series V was to determine the effect of omitting cement from a concrete mixture designed for a six-bag cement constant and to provide data for deriving an equation appropriate for predicting potential minimum 28-day strength from the results of early tests under these conditions.

In addition to these primary objectives, the designed experiment included a provision for use of data from series V for determining the effects of substituting pressure pads for sulfur mortar caps when early tests are made. The data were also intended for use in testing the fit of equation 1 to tests made at early ages without pretreatment of the specimens to accelerate the gain of early strength.

Specimen Preparation

The preparation, curing, and testing of concrete test specimens were conducted by the Materials Control, Soil and Testing Division of the West Virginia Department of Highways.

The order of making batches was pseudorandom so that there was no consistent pattern of a batch with a certain cement content being made after a batch with another cement content was made. Mix proportions are given in Table 1.

After mixing, the concrete from all batches was tested for slump and air content. Concrete used for these purposes was discarded. Cylinders were made by the group method in cardboard molds outside the moisture room and were then carried to the

Table 1. Mix proportions for series V.

Cement Constant	Cement, Marquette Type 1	Fine Aggregate, Ohio River Sand	Coarse Aggregate, No. 67 Acme Limestone	City Water	Air Content, Master Builders Vinsol Resin (percent)
Three bags	282	1,379	1,745	197 ± 7	7.5 to 10.4
Four bags	375	1,379	1,745	226 ± 4	5.5 to 8.1
Five bags	470	1,379	1,745	237 ± 3	5.7 to 7.5
Six bags	564	1,283	1,690	265 ± 3	5.2 to 5.8

Note: Values are in pounds per cubic yard. 1 lb/yd^3 = 0.59 kg/m^3.

moisture room for curing at standard temperature of approximately 73 F (23 C). All cylinders were tested without pretreatment and at storage temperature. One-half the cylinders were prepared for test with sulfur mortar (Atlas Vitrobond) caps. The other half were prepared by placing Celotex pressure pads ½ in. (12.7 mm) thick by 7 in. (178 mm) square on both ends of cylinders at time of test.

Companion cylinders were made from 30 batches, and 120 comparisons were made of the strengths obtained on 1 to 3-day-old specimens. The average strengths of specimens made from the same batch and the standard deviations are given in Table 2. The average strengths obtained by using sulfur mortar caps and pressure pads are given in Table 3. The overall reduction in strength due to the use of the pressure pads was less than 5 percent; most of the reduction in strength occurred in specimens testing in excess of 1,800 psi (12 000 kPa) as shown in Figure 1. Variation in strength, as measured by the standard deviation of tests within batches, was approximately the same for the Celotex pressure pads as for the sulfur mortar caps over the entire range of strengths. Because of the small differences, the two sets of data were pooled for some purposes.

Prediction Equation 2

For predicting minimum potential compressive strength at 28 days, equation 1 simplifies to

$$S_{28} = S_a + b\ (4.699 - \log m) \qquad (2)$$

where 4.699 is the log of the maturity of the concrete specimens stored for 28 days at approximately 73 F (23 C). The other parameters are the same as for equation 1 except that in the previous work where the specimens were conditioned by heating in a 200 F (93 C) water bath, it was necessary to determine a constant C, which accounted for the effects of heating and other possible undefined variables in equations 1 and 2. In series V, where no specimens were heated, the use of the constant C was not required for practical applications. The value of b was found by simple regression and by substituting actual compressive strength values and actual maturities based on temperature measurements in the equation $S_M = b\ (\log m) - a$.

The graphical relationship of maturity (F hour) to compressive strength [psi (kPa)] of the concrete at early ages and at 28 days for series V is shown in Figure 2. As shown in the figure, equation 2 is capable of making reliable predictions of minimum 28-day strengths providing that the temperature history (maturity) and the cement content of the mixture are known.

Comparison of Measured and Predicted Compressive Strengths

A comparison was made of measured 28-day compressive strengths [psi (kPa)] deter-

Table 2. Compressive strength of test specimens.

Cement Constant	Measurement	24 Hours SM[a]	24 Hours PP[b]	48 Hours SM[a]	48 Hours PP[b]	72 Hours SM[a]	72 Hours PP[b]	28 Days SM[a]
Three bags	Mean	273	282	503	458	657	670	1,572
	Standard deviation	45	48	61	52	27	19	205
	Coefficient of variation	16.4	18.2	12.1	11.1	4.2	2.8	13.0
Four bags	Mean	616	620	974	986	1,279	1,278	2,794
	Standard deviation	141	34	142	134	109	202	421
	Coefficient of variation	22.9	5.5	14.6	13.6	8.5	16.8	15.1
Five bags	Mean	934	919	1,402	1,441	1,717	1,776	3,831
	Standard deviation	142	73	110	77	182	102	247
	Coefficient of variation	15.2	7.9	7.8	5.3	10.6	5.8	6.4
Six bags	Mean	1,631	1,566	2,446	2,339	2,894	2,673	5,182
	Standard deviation	149	161	180	174	203	287	231
	Coefficient of variation	9.1	10.3	7.4	7.4	7.0	10.7	4.5

Note: Values are in pounds per square inch. 1 psi = 6.9 kPa.
[a]Sulfur mortar caps. [b]Celotex pressure pads.

Table 3. Average compressive strengths of concrete cylinders with sulfur mortar caps and pressure pads.

Sulfur Mortar Caps	Pressure Pads	Distribution of Differences: Amount	Distribution of Differences: Percent[a]	Sulfur Mortar Caps	Pressure Pads	Distribution of Differences: Amount	Distribution of Differences: Percent[a]
273	265	-8	-2.9	1,717	1,776	+59	+3.4
503	468	-35	-7.0	1,758	1,653	-105	-6.0
616	620	+4	+0.7	2,057	1,955	-102	-5.0
657	670	+13	+2.0	2,448	2,339	-109	-4.5
934	919	-15	-1.6	2,632	2,570	-62	-2.4
974	986	+12	+1.2	2,894	2,673	-221	-7.6
1,301	1,193	-108	-8.3	3,001	2,821	-180	-6.0
1,402	1,441	+39	+2.8	3,133	2,990	-143	-4.6
1,631	1,566	-65	-4.0	3,686	3,210	-476	-12.9
				$\bar{X}$ = 1,756	1,673	-83	-4.7
				σ = 1,021	928	93	-9.1

Note: Results are based on averages of six or seven test results. Values are in pounds per square inch. 1 psi = 6.9 kPa.
[a]Of sulfur mortar caps.

Figure 1. Compressive strength results for sulfur mortar caps versus pressure pads.

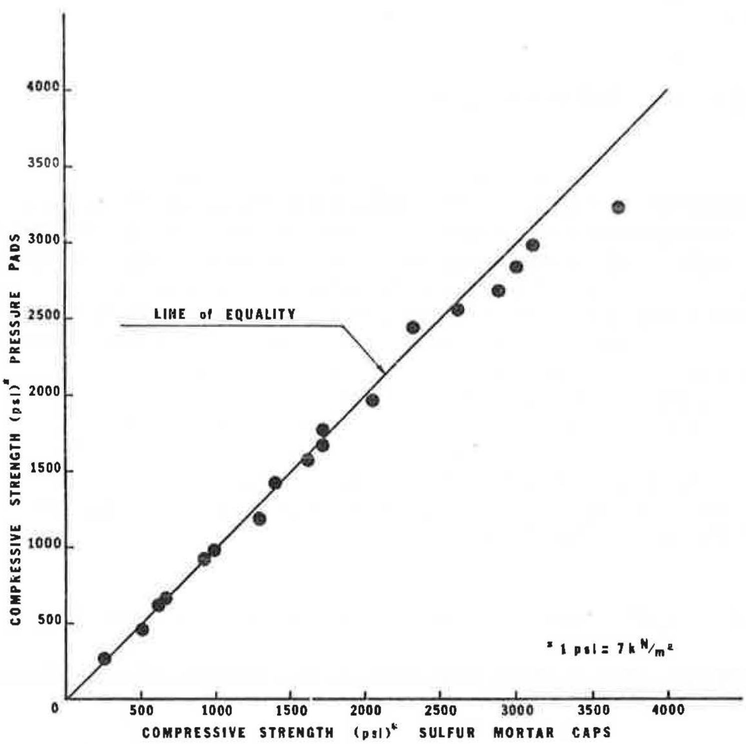


Figure 2. Early and 28-day compressive strengths based on equation 2.

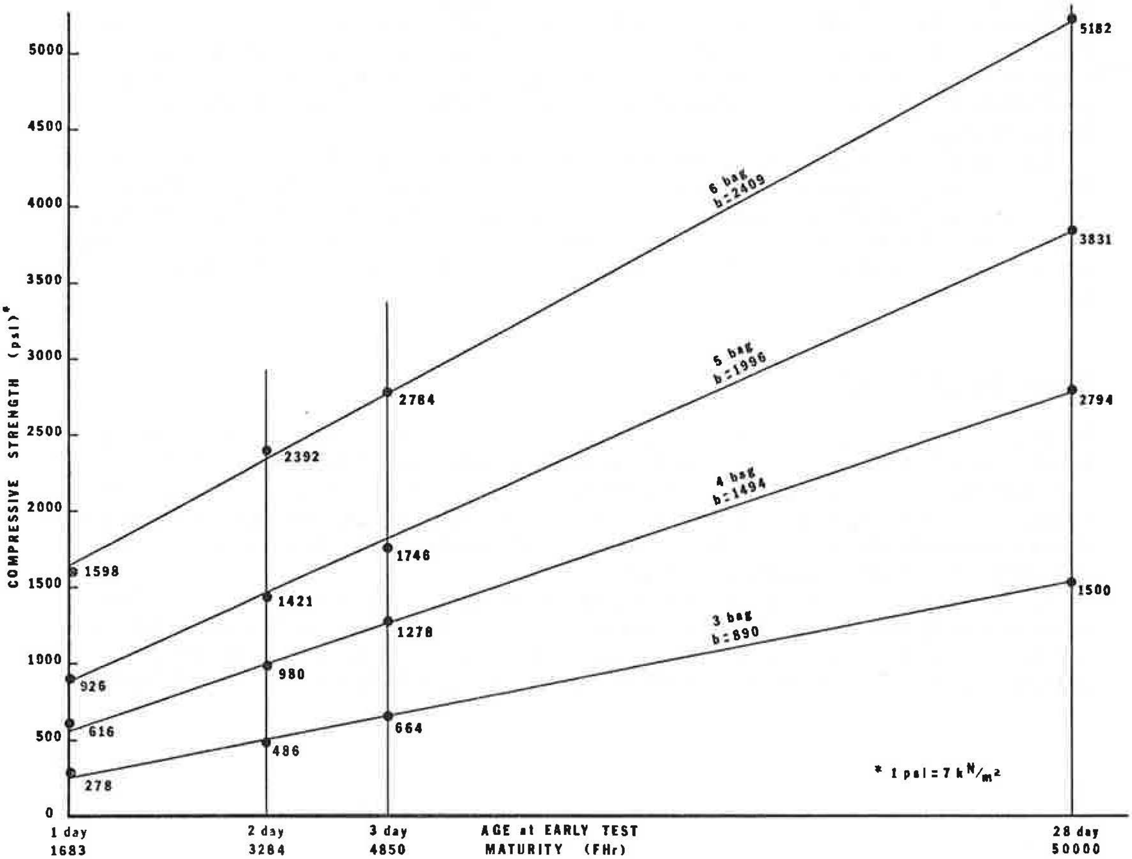
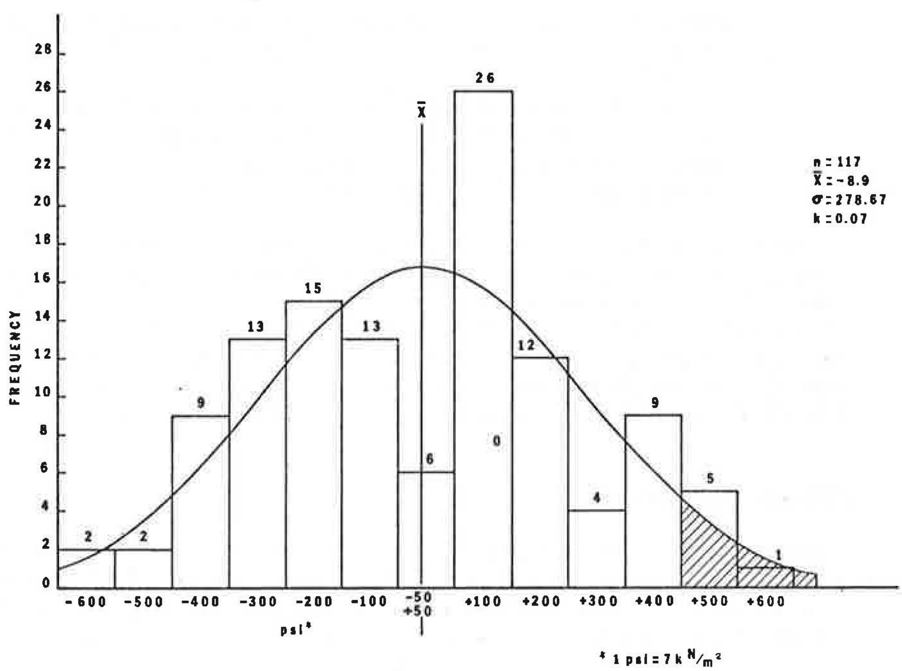


Figure 3. Frequency distribution of differences for values predicted by equation 2 and measured values.



mined in series V and the strengths estimated by equations 1 and 2. The distribution of the differences Δ between the measured values and the predicted values is shown in Figure 3. This shows that, if the predicted 28-day strength (S_{28}), computed by the correct form of equation 2, exceeds the critical acceptance value L by 1.645 σ_e or 460 psi (3200 kPa), there is a 95 percent probability that the true average strength is greater than L.

The standard deviation σ_e of differences between measured 28-day strengths and the strengths predicted by equations 1 and 2 from the results of tests of specimens not conditioned by heating is about 278 psi (1900 kPa). This is about 45 percent less than the σ_e value of 493 psi (3400 kPa) obtained from the results of tests of specimens conditioned by heating to 200 F (93 C) for 3½ hours as reported for the previous work (1).

Prediction Equation 3

As stated in the objectives, the primary purpose of series V was to obtain data that could be used to derive an equation that would predict the potential strength of concrete mixtures deficient in cement from the results of early tests. Equations 1 and 2 were satisfactory for predicting the potential 28-day strength of concrete made with a known cement constant but were unreliable when low strengths were due to a significant deficiency of cement of unknown amount.

In studies made to determine the possibility of developing a prediction equation, alternate to equations 1 and 2, it was found that results of early tests of concrete at a particular maturity were highly correlated with 28-day compressive strengths (9). The general form of the equation of relationship is for a parabola and can be written as

$$S_{c_{28}} = a\,(S_{c_m})^b \tag{3}$$

where

$S_{c_{28}}$ = compressive strength in psi (kPa) of specimens cured for 28 days under standard conditions and prepared for test by means of sulfur mortar caps,

a = value of the intercept of the line of relationship with the Y ordinate on a log-log plot,

S_{c_m} = compressive strength in psi (kPa) of specimens tested at a particular age or degree of maturity without pretreatment and prepared for test with pressure pads in place of sulfur mortar caps, and

b = slope of the line of relationship on a log-log plot.

As shown in Figure 4, equation 3 can be used to predict minimum 28-day strengths independently of cement content, providing that the early-age specimens are tested after 24, 48, or 72 hours of curing at about 73 F (23 C). The distribution of the differences between the measured values and the values predicted by equation 3 is shown in Figure 5. This figure shows that, if the predicted 28-day strength exceeds the critical acceptance limit L by 1.645 σ_e or 570 psi (3900 kPa), there is a 95 percent probability that the true average strength is greater than L.

Prediction Equation 4

Since equations 1 and 2 appeared to be more suitable for predicting potential 28-day compressive strength of concrete with a known cement content over a range of maturities, as shown in Figure 2, and equation 3 was suitable for prediction of potential 28-day strength of concrete when tested at a particular maturity, as shown in Figure 4,

Figure 4. Early and 28-day compressive strengths of concrete based on equation 3.

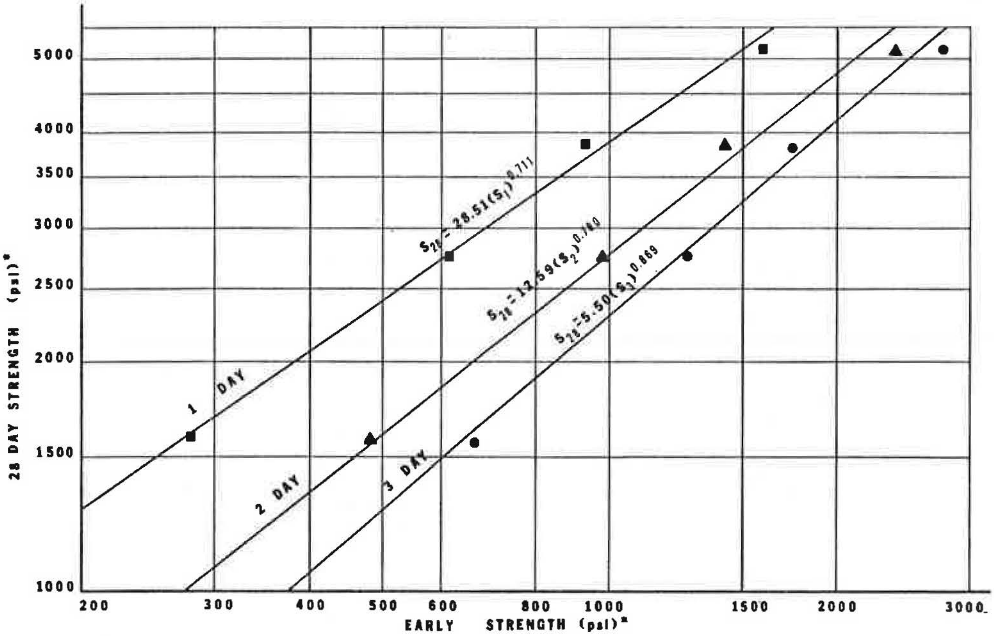


Figure 5. Frequency distribution of differences for values predicted by equation 3 and measured values.

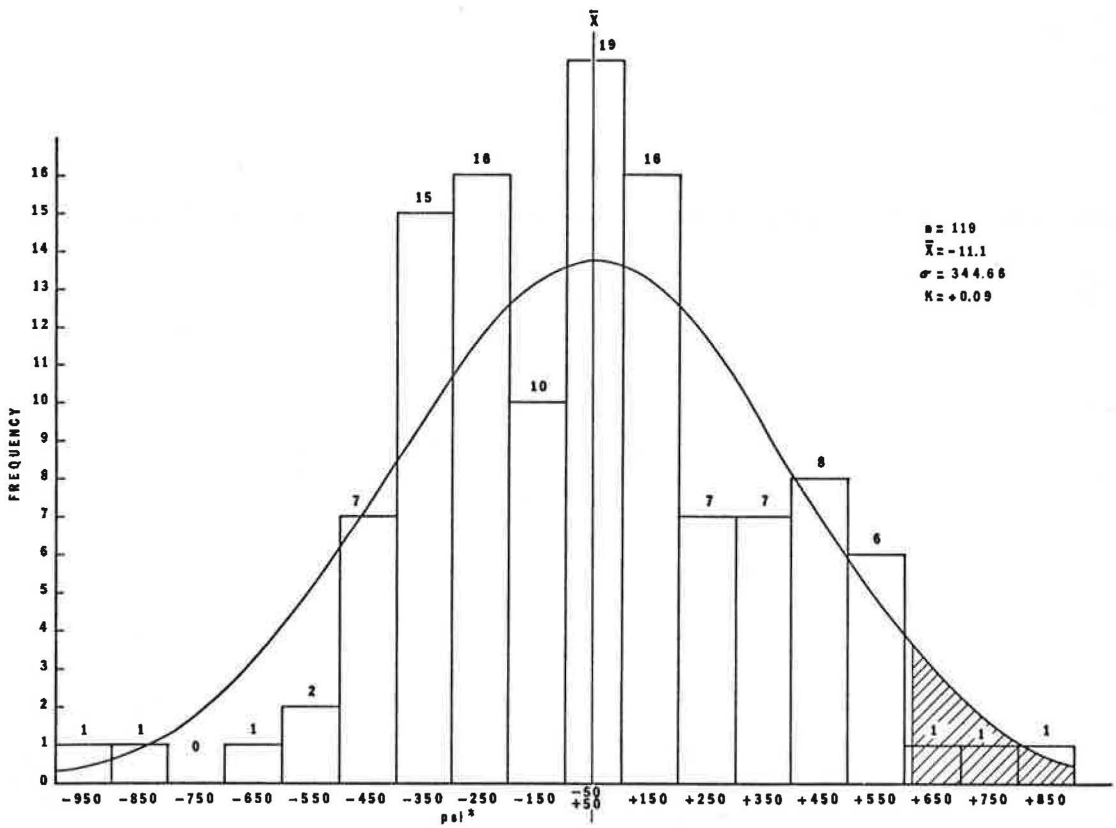


Figure 6. Nomograph for estimating potential 28-day compressive strength from results of early tests.

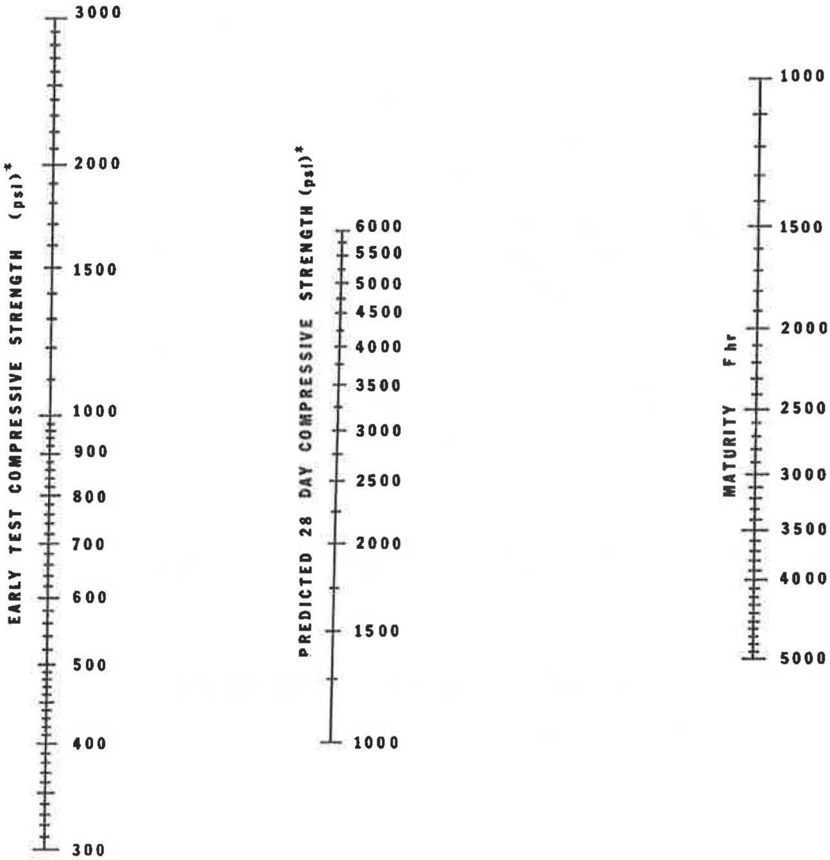
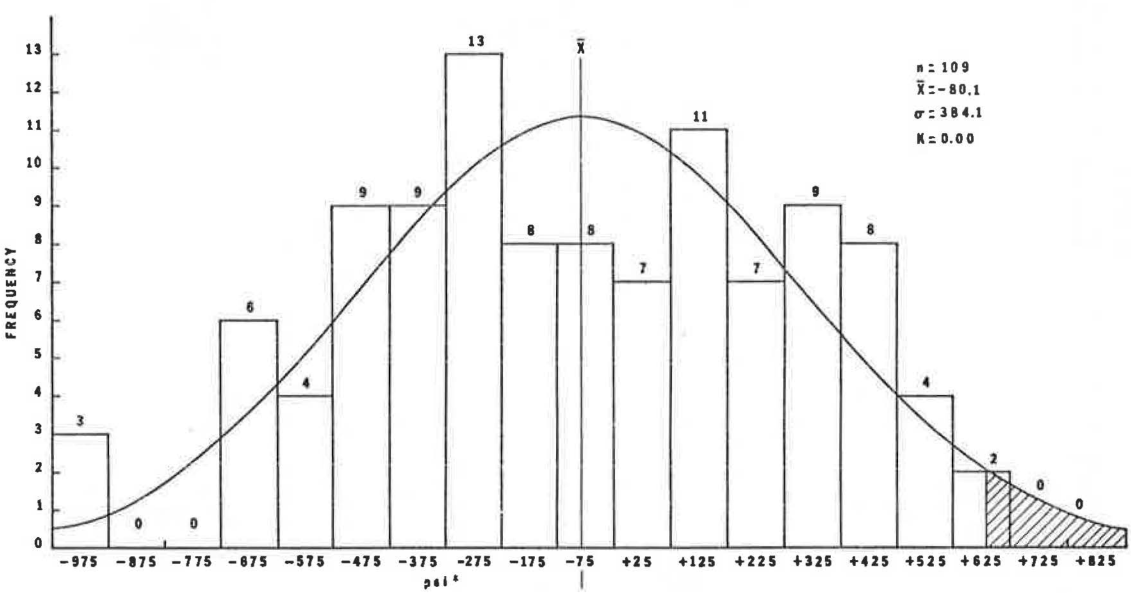


Figure 7. Frequency distribution of differences between values predicted by equation 4 and measured values.



the three equations were combined to provide an equation of more general application. This equation has the form $\log S_{28} = 2.9844 + 0.75 \log S_e - 0.51 \log m$, which may be written as

$$S_{28} = 965 \frac{S_e^{0.75}}{m^{0.51}} \tag{4}$$

where

S_{28} = predicted 28-day compressive strength,
S_e = compressive strength of specimens tested at an early age and having a maturity m, and
m = degree hours of maturity at the time of the test.

Equation 4 is independent of the cement content of the concrete for three to six bags/ yd^3 = (3.9 to 7.8 bags/m^3), and the early-strength specimens do not have to be tested at any particular maturity. Equation 4 can be solved by using the log scales on a slide rule, but, for frequent use, a nomograph similar to that shown in Figure 6 can be constructed.

This nomograph was used to compute predicted values for comparison with measured values. The distribution of the differences is shown in Figure 7. The standard deviation of the differences is about 384 psi (2600 kPa); there is a 95 percent probability that the true average strength exceeds the value of L.

Equations 1, 2, 3, and 4 illustrate types of mathematical relationships that can be used to predict minimum potential 28-day strength of concrete under particular conditions. Similar equations, with different parameters, can be developed for concrete mixtures with proportions and kinds of cement and aggregate other than those used in the mixtures reported here. The accuracy of the predictions will depend largely on the use of correct and uniform procedures in the making of the early and 28-day test specimens.

CONCLUSIONS

The findings of this experiment indicate the following:

1. Heat treatment or autogenous curing of concrete specimens that are more than about 20 hours old before early testing is not necessary and may complicate the derivation of equations predicting potential minimum 28-day strength. Testing cylinders after 1 to 3 days of curing at normal temperatures can effect savings in equipment expenditures and technical effort.
2. The use of sulfur mortar caps to prepare cylinders for test at early ages is not required. The use of cardboard or Celotex pressure pads for specimens expected to attain less than about 2,000 psi (13 800 kPa) can effect savings in material and equipment costs. When combined with testing without prior conditioning, the use of pressure pads greatly increases the practicality of the use of early compressive tests in the field as one means of quality determination.
3. When definite early test procedures are established and a series of laboratory tests have been performed, equations can be derived for predicting the minimum potential strength of concrete at 28 days. A suitable equation can predict minimum potential 28-day strength with sufficient accuracy to provide a valuable method for use in quality assurance systems.

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The contents of this report reflect the view of Materials Research and Development and the Materials Control, Soil and Testing Division of West Virginia Department of Highways, which are responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official view or policies of the sponsoring agencies. This report does not constitute a standard, specification, or regulation.

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CANADIAN EXPERIENCE IN THE USE OF THE MODIFIED BOILING METHOD

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This paper briefly reviews the history of accelerated-curing techniques for quality control of concrete. In a modified boiling method that has been developed by the Mines Branch, the test specimens are standard cured for 23 to $23\frac{1}{2}$ hours, then boiled for $2\frac{1}{2}$ hours, and tested for compression $1\frac{1}{2}$ hours after being boiled. The total elapsed time between molding and testing of cylinders does not exceed 28.5 hours. This method is finding increasing acceptance in Canada and has been adopted by the American Society for Testing and Materials and the Canadian Standards Association. The data from Hydro-Quebec, Acres-Bechtel, a ready-mix concrete company in Ottawa, and the University of Calgary, Alberta, show significant correlations between the results of the accelerated and 28-day compressive strength tests. The original regression equation reported by the Mines Branch relating the compressive strength of accelerated-cured cylinders to that of 28-day standard-cured cylinders has been compared with the equation developed at the University of Calgary, and the two regression lines are about identical. Each testing and control authority contemplating the use of the modified boiling method is urged to develop its own correlations for predicting 28-day compressive strengths and not to rely on the correlations published by others.

•RAPID advances have been made in the processing and manufacture of mineral aggregates and cements during the past 50 years. Unfortunately, little of the progress is reflected in the techniques used for quality control of concrete in the field. The concrete industry is still burdened with control tests in which cylindrical specimens are subjected to a compression test at 28 days. This waiting period is anachronistic when multistory buildings are being completed in a matter of months.

Notwithstanding the fact that the compression test has successfully served the construction industry for the past several decades, the faster pace of modern-day construction practices requires new quality control tests in which the waiting period of 28 days is eliminated or considerably reduced. This paper traces the efforts of earlier researchers to develop a rapid test to control the quality of concrete and describes, in some detail, a new modified boiling procedure that is becoming increasingly popular in Canada and several other countries.

BACKGROUND

Despite the perennial problems associated with the 28-day compression test, efforts to develop rapid tests for quality control of concrete are recent. The pioneering work in the field of accelerated-strength testing of concrete was done in the United States between 1925 and 1935. One of the earliest publications on the subject was in 1927 by Gerend (1), who showed that a rapid gain in concrete strength was achieved by subjecting 6 by 12-in. (152 by 305-mm) cylinders to a saturated steam bath at 80 to 100 lbf/in.^{2} (552 to 689 kPa). The need of an autoclave to increase the rate of strength gain was an obvious drawback, and the procedure failed to gain acceptance.

The construction of the Hoover Dam in the 1930s caused the U.S. Bureau of

Reclamation to investigate the use of an 8-hour, boiling-water accelerated strength test (2). The boiling of 6 by 12-in. (152 by 305-mm) cylinders that had been molded in special jackets was limited to 7 hours, and 1 hour was used for cooling, capping, and testing the specimens. After several years of field trials, the test procedure was considered unsatisfactory because of the lack of precision in the predicted 28-day compressive strengths.

From the late 1930s until the early 1950s, no significant progress in the development and use of accelerated strength testing of concrete was reported anywhere in the world. At that time, a number of research papers dealing with rapid control tests originated in the United Kingdom (3, 4, 5). Then, in 1963, the Réunion International des Laboratoires d'Essais et de Recherches sur les Matériaux et les Constructions in Paris sponsored an international correspondence symposium on the theme of accelerated hardening of concrete and rapid control tests (6). In North America, Canada has been in the forefront of the development of accelerated strength tests. Since 1962, the Mines Branch of the Ottawa Department of Energy, Mines and Resources has made significant contributions in this field (7, 8, 9, 10, 11, 12).

MODIFIED BOILING METHOD

Malhotra et al. (7) adopted the modified boiling method originally proposed by Akroyd (4) for further study and development and used 6 by 12-in. (152 by 305-mm) cylinders. The salient features of this technique, which has now been accepted by the American Society for Testing and Materials (ASTM C 684-73 T) and the Canadian Standards Association (CSA A 23.2.26), are as follows:

1. Prepare two 6 by 12-in. (152 by 305-mm) test cylinders in steel molds using standard molding methods. The delay between mixing concrete and preparing the test specimens should not exceed 30 min.
2. Immediately after the cylinders are molded, close all molds tightly with steel cover plates and place them in a moist-curing room or in a box maintained at about 73 ± 3 F (23 ± 1.7 C) and 100 percent relative humidity. If suitable moist-curing facilities are not available, cover the molds with wet burlap and keep them wet for 23 hours.
3. At the end of the curing period, place the cylinders, complete with molds and covers, in boiling water. Keep the temperature of water just below the boiling point to avoid excessive evaporation.
4. After 3.5 hours of boiling, remove the cylinders from the boiling water, strip the molds, and allow the specimens to cool for about one-half hour.
5. Weigh the test cylinders, cap them, and test them in compression 1 hour later.

The total elapsed time between molding and testing of the test cylinders is 28.5 hours.

FIELD USE OF MODIFIED BOILING METHOD

Since publication of the results of the original investigation by Malhotra et al. (7), the modified boiling method has been adopted by a number of organizations in Canada and elsewhere for routine quality control of concrete (9, 10, 11, 12, 13). The method is probably widely accepted because it is simple and the curing cycle can be controlled easily.

Test data from a cross section of organizations that have used this method are shown in Figures 1 (9), 2 (10), 3 (9), 4 (11), and 5 (12). The regression equations are also shown where they were available. The test results for lightweight concrete are shown in Figure 6 (13).

Figure 1. Field data for accelerated versus 28-day compressive strength for nine jobs from coast to coast in Canada using normal portland cement concrete.

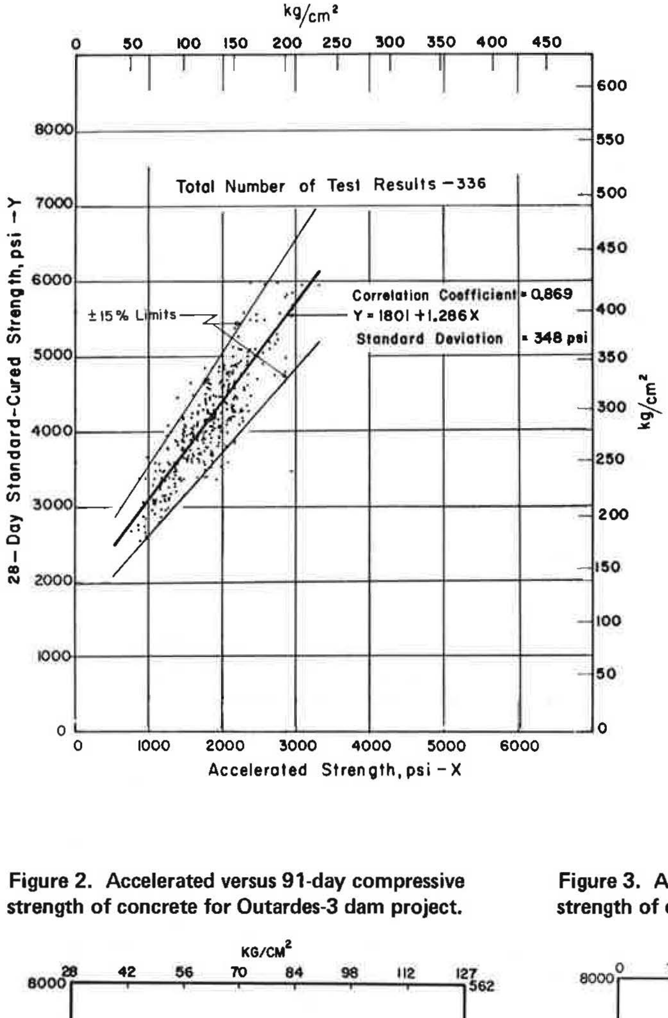


Figure 2. Accelerated versus 91-day compressive strength of concrete for Outardes-3 dam project.

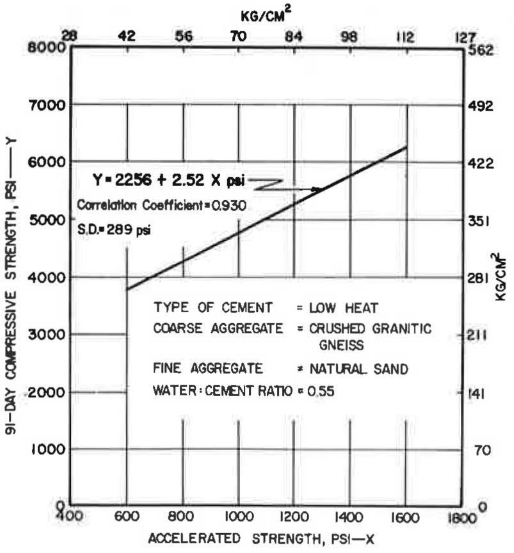


Figure 3. Accelerated versus 28-day compressive strength of concrete for Churchill Falls project.

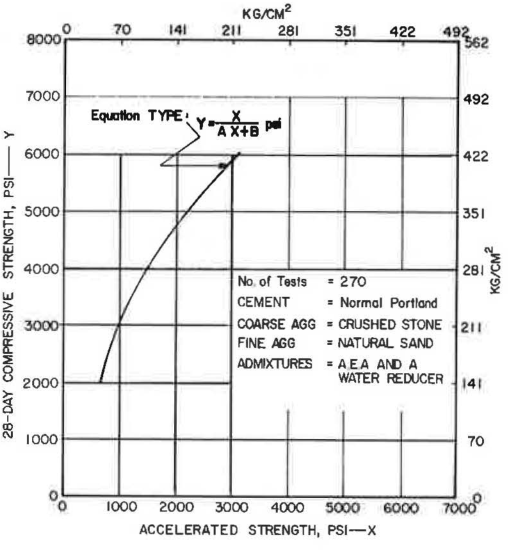


Figure 4. Accelerated versus 28-day compressive strength of ready-mix concrete.

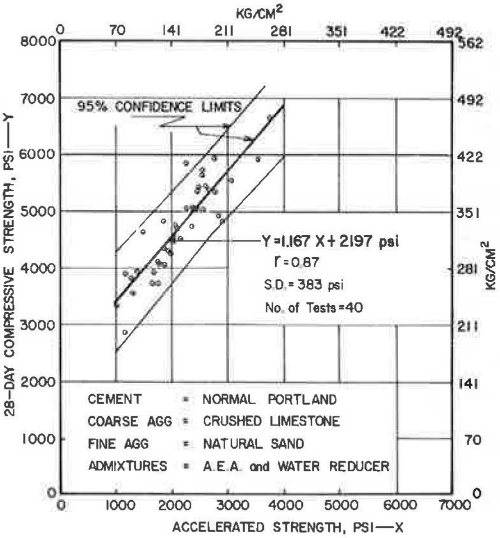


Figure 5. Accelerated versus 28-day compressive strength of concrete for a project in Calgary.

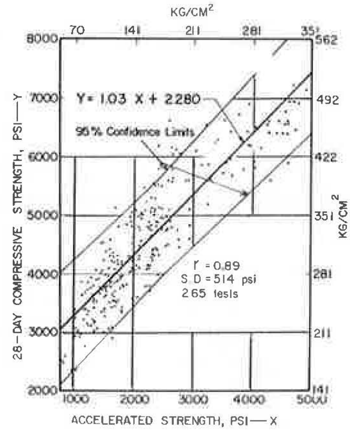


Figure 6. Accelerated versus 28-day compressive strength for lightweight concrete.

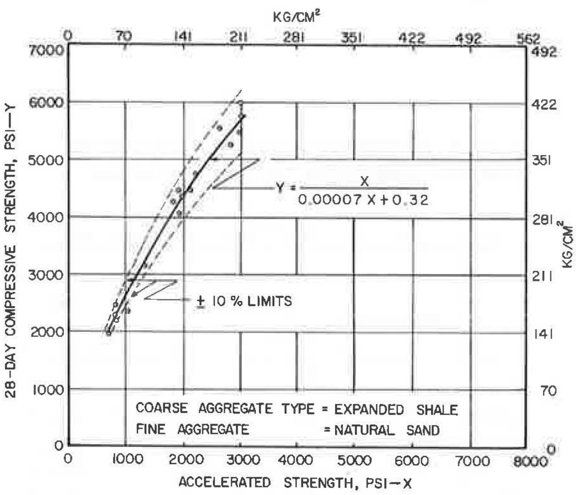
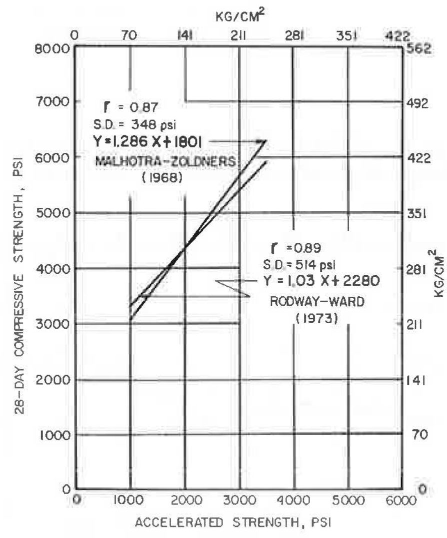


Figure 7. Accelerated versus 28-day compressive strength of concrete from different research projects.



GENERAL COMMENTS ON FIELD USE OF MODIFIED BOILING METHOD

To date, the modified boiling method has primarily been used for trouble shooting and quality control testing rather than for acceptance testing. The plots of test data (Figures 1, 2, 3, 4, 5, and 6) indicate that the modified boiling method can satisfactorily predict the 28-day compressive strength of concrete. The available correlation coefficients are greater than 0.86, indicating that correlations are significant. The test method is equally applicable to normal and to lightweight concretes. The degree of correlation does not appear to be affected by the use of high-early-strength and low-heat cements.

The original regression equation by Malhotra and Zoldners (9), relating the compressive strength of accelerated-cured cylinders to that of 28-day standard-cured cylinders, has been compared with the equation developed by Rodway and Ward (Figure 7). It is gratifying to note that the two regression lines are about identical.

Despite the high degree of correlation, however, each testing and control authority contemplating the use of the modified boiling method is urged to develop its own correlations for predicting 28-day compressive strengths and not to rely on the correlations published by others.

CONCLUDING REMARKS

The steadily increasing pace of modern construction demands the use of rapid evaluation techniques to control the quality of concrete in construction. ASTM and CSA have now published standard accelerated strength tests. Unfortunately, these tests, including the modified boiling method, are still not rapid enough because they require a waiting period of 24 to 48 hours; however, they are a vast improvement over the previous waiting period of 7 or 28 days. It is hoped that accelerated tests, instead of the 28-day test, will soon form the basis of design and the acceptance criterion and that researchers will ultimately find a method for determining the potential strength of concrete immediately after it has been mixed.

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PREDICTION OF STRENGTH DEVELOPMENT IN CONCRETE STRUCTURES

B. M. Sadgrove,* Construction Industry Research and Information Association

Early removal of side forms from concrete members may cause mechnical damage or lead to frost damage. Currently, achievement of a cube compressive strength of 5 N/mm^2 is the criterion for avoiding such damage, but recent research indicates that the value may be reduced to 2 N/mm^2. Soffit forms may be removed when members have sufficient flexural strength to withstand self-weight of the concrete and other applied loads. In addition, deflections must not be excessive. Research has shown that the proportion of design load applied at early age should not exceed the proportion of design compressive strength achieved at that age in the member under consideration. The relationships between compressive strength and two parameters, maturity equivalent and equivalent age, are discussed. Equivalent age is shown to be useful for the prediction of strength from a known temperature history at early age when the compressive strength development is very sensitive to curing temperature. At later ages, maturity equivalent may be more suitable for prediction purposes. A method of prediction of temperature history in hydrating concrete members has been combined with these methods of prediction of compressive strength to determine the times at which formwork may be removed, based on the criteria described.

•THE material cost of the formwork is a substantial part of the overall cost of a reinforced concrete structure. In addition, delays in its reuse may lead to late completion of the structure and increased on-site costs. Considerable economy may therefore be achieved by removing the formwork from a structural member and reusing it as soon as possible.

The criteria for striking formwork depend on whether the formwork is used vertically or horizontally, that is, as side forms or soffit forms. Side forms are only required to contain the concrete until such time as it is self-supporting. Thereafter, they may be removed subject to avoiding damage in the process of removal and damage caused by freezing of immature concrete in adverse weather conditions. Soffit forms, however, must remain in place until the structural member is capable of carrying its self-weight and any applied construction loads without risk of collapse or excessive deflection.

The criteria for these situations, as discussed in this paper, may be related to the development of the compressive strength of the concrete. This, in itself, depends primarily on the concrete mix proportions and its temperature history. The latter is, of course, determined by a number of factors including the dimensions of the structural member, the thermal insulation value of the formwork, the ambient temperature, the placing temperature of the concrete, and the cement type and content.

A method of temperature prediction that considers these parameters is available. Provided that a suitable means can be found to relate the compressive strength of concrete to its temperature history, the compressive strength development in a structural member and hence formwork striking times may be predicted.

*Mr. Sadgrove was with the Cement and Concrete Association, London, when this research was done.

Frost Damage

A number of researchers have reported that freezing concrete at early age may lead to a considerable loss of potential strength. Möller (1) summarized earlier work and conducted his own experiments to determine at what age concrete is no longer susceptible to frost damage (as distinct from freeze-thaw damage). Both he and Powers (2) concluded that the required ages for concrete of various design strengths may be conveniently related to the achievement of a given compressive strength. This conclusion seems to have been the basis of various specification clauses. For example, the British Code of Practice CP110 specifies that the temperature of concrete is unlikely to be damaged when it has reached a cylinder strength of 3.4 N/mm^2. Considering the correlation factor between cube and cylinder strengths, these two criteria are in fairly close agreement.

A point that must be emphasized is that these strength criteria are not indicative of the ability of concrete to resist tensile forces created by the expansion that takes place during the phase change of water to ice. They are related, as Powers (2) has suggested, to a reduction in free water and the creation of pore space to accommodate ice as it forms.

Sadgrove (3) has reexamined the strength criterion by determining the loss of strength that occurs at 28 days and 91 days as a result of early freezing. He concluded that minimal loss of strength would prevail if the concrete had achieved a cube strength of 2 N/mm^2 before freezing. This lower strength criterion may permit a modest reduction in striking times.

Mechanical Damage

Before the work by Harrison (7), there seems to have been no experimental work reported on this subject; an unsupported recommendation is given elsewhere (5). Harrison examined the damage caused by early striking of formwork to the surfaces and arrises of concrete prisms. He concluded that the risk of damage was slight provided that the concrete had achieved a cube strength of 2 N/mm^2 for all but the very high-strength mixes. For the latter, a minimum curing period was necessary. These recommendations assumed that reasonable care was taken in removing the formwork. Subsequent mechanical damage caused, for example, by site plant cannot be avoided by a strength criterion; physical means of protection must be used.

Dead and Imposed Loading

Loads applied to beams or slabs at early age may cause failure through crushing of the concrete or inadequate bond. Alternatively, the loads may be adequately supported but give rise to excessive deflections. In determining the proportion of load that may be applied at early age, Sadgrove studied the development of bond strength (6) and the structural strength and deflection characteristics of beams loaded at early age (7).

From theoretical considerations, the ultimate moment of a balanced section, at any age up to 28 days, should be directly proportional to the compressive strength of the concrete at that age as shown in Figure 1 (1), and an underreinforced section will develop flexural strength more rapidly at early ages. This has been confirmed experimentally (7). Bond strength was also found to develop more rapidly than compressive strength at early age (6). Thus it was shown that bond was unlikely to be a limiting criterion for early loading if the assumption is made that all sections are balanced.

The shrinkage, elastic, and creep deflections at early age were measured (7). The shrinkage deflection was independent of the age at which drying began. Elastic and creep coefficients were much greater at early age. However, the limitation on the load that can be applied at early age and the rapid decrease in the coefficients with

Figure 1. Development of ultimate moment of resistance, bond strength, and compressive strength.

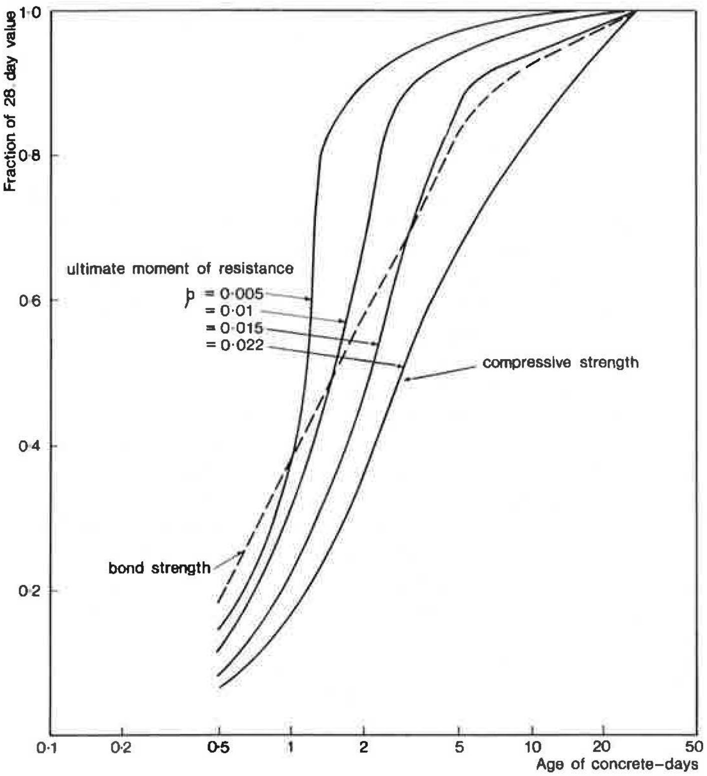
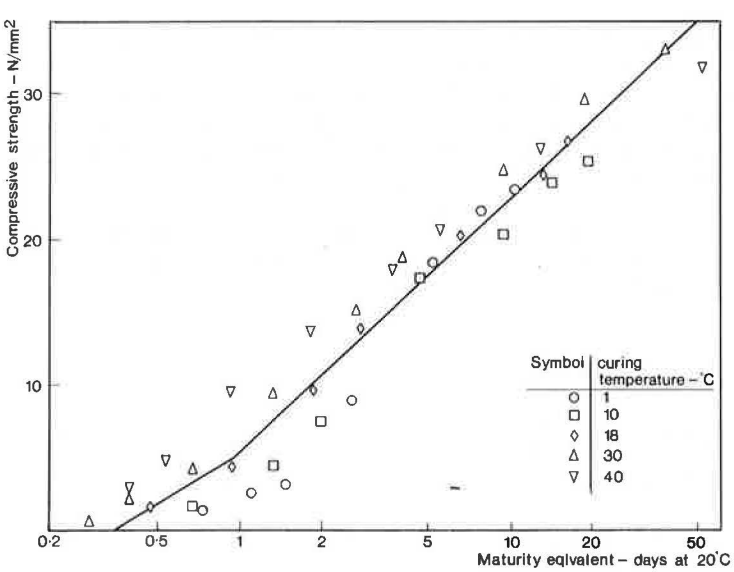


Figure 2. Compressive strength versus maturity equivalent.



increasing age effectively caused no significant increase in the deflections that might be anticipated at later age with normal loading patterns.

The final conclusion is that the proportion of full working load that may be applied to a structural member at early age is equal to the proportion of the design strength of the concrete achieved at that age.

DEVELOPMENT OF COMPRESSIVE STRENGTH

A number of researchers (10, 11, 12, 13) have proposed that there is a relationship between the compressive strength of concrete and maturity that is defined as $\Sigma(\theta + 10)\Delta t$, where θ = temperature of the concrete at any time and Δt = increment of time.

It is postulated that samples of the same concrete having equal maturities will have equal compressive strengths, regardless of their temperature histories. The results from various series of tests on the compressive strength of concretes cured over a range of temperatures have, in general, validated this hypothesis, but most of the results considered have related to fairly mature concrete. Results have been reported by Sadgrove (7) on concrete cured at constant temperature in the range of 1 to 45 C and tested in compression at ages of 5 hours to 28 days. When the compressive strength values from these tests were plotted against maturity, it was found that, although there was quite a good relationship for comparatively mature concrete, there was considerable scatter at low values of maturity. This is shown in Figure 2, where compressive strength is plotted against maturity equivalent, that is, simply maturity divided by 30 C and expressed as days at 20 C. Examination of the pattern of the results at low maturity equivalent indicated that compressive strength is initially more sensitive to changes in curing temperature than maturity suggests. A trial and error method was therefore used to determine the factor related to temperature by which actual age is related to age at 20 C. The factor finally proposed is

$$F = \left[\frac{\theta + 16}{36}\right]^2 \qquad (1)$$

where θ is not less than -10 C.

The sum of the products of this factor and increments of time, that is, $\Sigma F\Delta t$, is called equivalent age and is expressed in days at 20 C.

The values of compressive strength shown in Figure 2 are replotted against equivalent age in Figure 3. Much better agreement is now evident at strengths up to about 15 N/mm^2, and agreement beyond this level is fairly good. However, a given error in equivalent age gives a larger difference in actual time than the same error in maturity equivalent because the former depends on a second-order equation of temperature. Equivalent age therefore appears to be most suitable as a parameter for the prediction of strength at early age; maturity (or maturity equivalent) may be more appropriate at later age (an arbitrary division might be made at 3 days equivalent age).

PREDICTION OF TEMPERATURE HISTORY

Weaver (12) has developed a method of predicting the temperature history of a hydrating concrete element with a constant ambient temperature based on biaxial heat flow. For a solution of the heat flow equation, values for the following are required:

1. Placing temperature of the concrete,
2. Ambient temperature,
3. Section dimensions,
4. Boundary conductances,
5. Thermal conductivity of the concrete,

Figure 3. Compressive strength versus equivalent age.

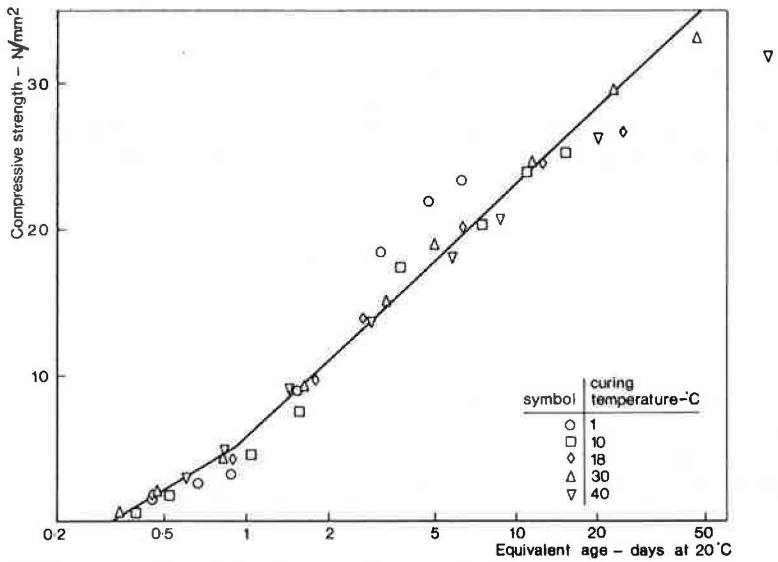


Table 1. Striking times of portland cement concrete with 330-kg/m^3 cement content and 22.5-N/mm^2 characteristic strength.

Formwork Conductance (W/m^2 deg C)	Ambient Temperature (C)	Resistance to Damage (N/mm^2)	Characteristic Strength Reached	
			33 Percent	66 Percent
18.9	-5	—[a]	—[a]	—[a]
	0	155	245	—[b]
	5	88	140	476
	10	60	93	309
	15	42	66	219
10.0	-5	—[a]	—[a]	—[a]
	0	140	230	—[b]
	5	80	132	464
	10	54	87	303
	15	39	60	213
6.5	-5	—[a]	—[a]	—[a]
	0	120	208	—[b]
	5	69	120	447
	10	48	78	292
	15	36	56	206
5.0	-5	—[a]	—[a]	—[a]
	0	105	192	—[b]
	5	62	110	434
	10	44	72	282
	15	35	53	199
3.4	-5	—[a]	—[a]	—[a]
	0	81	156	—[b]
	5	52	90	404
	10	40	62	261
	15	33	48	183
2.0	-5	68	156	—[b]
	0	48	81	570
	5	40	60	316
	10	35	50	201
	15	32	43	142
1.3	-5	48	76	—[b]
	0	42	60	429
	5	36	52	238
	10	34	47	163
	15	32	43	127

[a]Concrete is frozen before it reaches D level of 5 N/mm^2.
[b]Strength level is not reached before 1,000 hours.

6. Specific heat of the concrete,
7. Density of the concrete,
8. Thermal diffusivity of the concrete,
9. Cement content, and
10. Heat evolved by the cement in a given time increment.

The latter value may be predicted from data giving the total heat evolved at any time at a reference temperature and by using the time-temperature equation proposed by Rastrup (13):

$$\tau_r = 2^{\gamma(\theta - \theta_r)} \cdot t \tag{2}$$

where

θ = reaction temperature (here assumed constant),
θ_r = reference temperature,
t = time (from start of hydration),
τ_r = equivalent time at reference temperature, and
γ = reciprocal of the increase in temperature that doubles the rate of hydration.

The value of γ may be determined from the slope of the lines relating logarithm of time to evolve given total quantities of heat to the difference between reaction and reference temperatures. Thus heat evolution data are also needed at temperatures other than the reference temperature.

PREDICTION OF STRIKING TIMES

A computer program has been written, based on the method derived by Weaver (12), that predicts the temperature at the arris of a hydrating concrete element at given time intervals. The temperature at the arris is the lowest that is likely to occur within the element, and strength prediction based on the temperature at this point will therefore be conservative. The equivalent age increment of each time interval is calculated, and the total equivalent age is determined. This value is then compared with the equivalent age required for a given concrete to achieve various levels of compressive strength. When the required equivalent age is reached, the actual age is tabulated. Thus a series of striking times have been produced (14).

In addition to the values required for the temperature prediction as given in the previous section, the program must have input data relating to the concrete design strength and the strength levels for which times are predicted. The first set of tables uses design cube strengths of 22.5, 30, and 37.5 N/mm^2 because the tables were produced to conform with the draft Code of Practice (15). A revision is envisaged that would conform with the characteristic strengths specified in CP110, that is, 20, 25, 30 and 40 N/mm^2. The strength levels are D, 33 percent, and 66 percent, as given in Tables 1, 2, 3, 4, 5, and 6 (15). The placing temperature is 10 C. D indicates the level at which concrete is unlikely to suffer frost or mechanical damage, for which a strength of 5 N/mm^2 is used; the second and third levels are percentages of the characteristic strength. In the revised tables, 2 N/mm^2 would be used for the first level, and a fourth level equal to 100 percent of design strength may be introduced. However, for the 66 and 100 percent levels, the parameter maturity equivalent would probably be used in the prediction rather than equivalent age in view of the previous discussion.

A version of the striking times tables relating to current United States practice is being prepared. The units are in the U.S. customary system, and American Concrete Institute standards are used where relevant. Because of the wider range of ambient temperatures prevailing in the United States, the ranges have been selected as appropriate to the form type. For instance, very low ambient temperatures are only considered where high insulation values are used.

Table 2. Striking times of portland cement concrete with 380-kg/m³ cement content and 22.5-N/mm² characteristic strength.

Formwork Conductance (W/m² deg C)	Ambient Temperature (C)	Resistance to Damage (N/mm²)	Characteristic Strength Reached 33 Percent	66 Percent
18.9	-5	—[a]	—[a]	—[a]
	0	150	240	—[b]
	5	88	140	472
	10	57	93	309
	15	42	63	219
10.0	-5	—[a]	—[a]	—[a]
	0	135	225	—[b]
	5	80	128	460
	10	51	84	300
	15	36	60	210
6.5	-5	—[a]	—[a]	—[a]
	0	116	204	—[b]
	5	66	114	441
	10	44	74	286
	15	34	52	202
5.0	-5	—[a]	—[a]	—[a]
	0	99	183	—[b]
	5	58	104	426
	10	41	67	276
	15	32	49	194
3.4	-5	—[a]	—[a]	—[a]
	0	72	144	—[b]
	5	48	82	392
	10	37	57	250
	15	31	44	174
2.0	-5	56	116	—[b]
	0	42	69	525
	5	36	52	286
	10	33	45	179
	15	29	39	125
1.3	-5	40	64	—[b]
	0	36	51	357
	5	34	46	196
	10	31	42	138
	15	29	38	110

[a]Concrete is frozen before it reaches D level of 5 N/mm².
[b]Strength level is not reached before 1,000 hours.

Table 3. Striking times of portland cement concrete with 380-kg/m³ cement content and 30.0-N/mm² characteristic strength.

Formwork Conductance (W/m² deg C)	Ambient Temperature (C)	Resistance to Damage (N/mm²)	Characteristic Strength Reached 33 Percent	66 Percent
18.9	-5	—[a]	—[a]	—[a]
	0	115	240	—[b]
	5	64	140	452
	10	42	93	297
	15	30	63	210
10.0	-5	—[a]	—[a]	—[a]
	0	100	225	—[b]
	5	56	128	440
	10	36	84	285
	15	27	60	201
6.5	-5	—[a]	—[a]	—[a]
	0	80	204	—[b]
	5	48	114	420
	10	34	74	274
	15	26	52	192
5.0	-5	—[a]	—[a]	—[a]
	0	66	183	—[b]
	5	42	104	406
	10	32	67	262
	15	26	49	184
3.4	-5	—[a]	—[a]	—[a]
	0	51	144	657
	5	36	82	370
	10	30	57	237
	15	25	44	165
2.0	-5	40	116	—[b]
	0	36	69	489
	5	30	52	266
	10	27	45	167
	15	25	39	118
1.3	-5	32	64	—[b]
	0	30	51	321
	5	28	46	180
	10	26	42	129
	15	25	38	104

[a]Concrete is frozen before it reaches D level of 5 N/mm².
[b]Strength level is not reached before 1,000 hours.

Table 4. Striking times of portland cement concrete with 450-kg/m³ cement content and 30.0-N/mm² characteristic strength.

Formwork Conductance (W/m² deg C)	Ambient Temperature (C)	Resistance to Damage (N/mm²)	Characteristic Strength Reached: 33 Percent	Characteristic Strength Reached: 66 Percent
18.9	-5	—[a]	—[a]	—[a]
	0	110	240	—[b]
	5	64	140	452
	10	42	90	294
	15	30	63	207
10.0	-5	—[a]	—[a]	—[a]
	0	95	220	—[b]
	5	52	124	436
	10	36	81	282
	15	27	57	198
6.5	-5	—[a]	—[a]	—[a]
	0	72	196	—[b]
	5	42	108	414
	10	30	68	266
	15	24	48	186
5.0	-5	—[a]	—[a]	—[a]
	0	57	171	—[b]
	5	38	94	394
	10	29	60	253
	15	24	44	176
3.4	-5	—[a]	—[a]	—[a]
	0	42	123	633
	5	32	70	352
	10	27	49	222
	15	23	39	152
2.0	-5	36	76	—[b]
	0	30	54	420
	5	28	44	218
	10	25	38	133
	15	23	34	95
1.3	-5	28	48	576
	0	27	42	213
	5	26	40	130
	10	24	36	101
	15	23	33	85

[a]Concrete is frozen before it reaches D level of 5 N/mm².
[b]Strength level is not reached before 1,000 hours.

Table 5. Striking times of portland cement concrete with 450-kg/m³ cement content and 37.5-N/mm² characteristic strength.

Formwork Conductance (W/m² deg C)	Ambient Temperature (C)	Resistance to Damage (N/mm²)	Characteristic Strength Reached: 33 Percent	Characteristic Strength Reached: 66 Percent
18.9	-5	—[a]	—[a]	—[a]
	0	85	200	—[b]
	5	52	116	432
	10	33	75	282
	15	24	54	198
10.0	-5	—[a]	—[a]	—[a]
	0	70	180	—[b]
	5	40	104	416
	10	27	66	270
	15	21	45	189
6.5	-5	—[a]	—[a]	—[a]
	0	52	156	—[b]
	5	33	87	396
	10	24	56	256
	15	20	40	178
5.0	-5	—[a]	—[a]	—[a]
	0	42	132	663
	5	30	74	376
	10	24	49	242
	15	20	37	168
3.4	-5	52	220	—[b]
	0	33	90	600
	5	28	56	334
	10	23	41	211
	15	20	34	144
2.0	-5	32	60	—[b]
	0	27	45	393
	5	24	38	202
	10	22	34	124
	15	21	31	90
1.3	-5	24	44	512
	0	24	39	189
	5	22	34	120
	10	21	32	95
	15	20	30	81

[a]Concrete is frozen before it reaches D level of 5 N/mm².
[b]Strength level is not reached before 1,000 hours.

Table 6. Striking times of portland cement concrete with 490-kg/m³ cement content and 37.5-N/mm² characteristic strength.

Formwork Conductance (W/m² deg C)	Ambient Temperature (C)	Resistance to Damage (N/mm²)	Characteristic Strength Reached 33 Percent	Characteristic Strength Reached 66 Percent
18.9	-5	—[a]	—[a]	—[a]
	0	85	200	—[b]
	5	48	116	432
	10	33	75	282
	15	21	51	198
10.0	-5	—[a]	—[a]	—[a]
	0	70	180	—[b]
	5	40	100	416
	10	27	63	270
	15	21	45	189
6.5	-5	—[a]	—[a]	—[a]
	0	52	152	—[b]
	5	33	84	390
	10	24	52	252
	15	20	38	176
5.0	-5	—[a]	—[a]	—[a]
	0	39	126	654
	5	28	68	370
	10	23	45	236
	15	19	34	164
3.4	-5	48	200	—[b]
	0	33	81	585
	5	26	50	322
	10	22	38	201
	15	20	31	136
2.0	-5	28	52	—[b]
	0	24	42	345
	5	22	36	170
	10	21	32	105
	15	20	29	78
1.3	-5	24	40	356
	0	21	36	141
	5	22	32	100
	10	20	30	83
	15	19	28	72

[a]Concrete is frozen before it reaches D level of 5 N/mm².
[b]Strength level is not reached before 1,000 hours.

CONCLUSIONS

1. Methods of relating the compressive strength of concrete to time and temperature have been combined with a method of temperature prediction in hydrating concrete elements to predict the times at which certain levels of strength are reached in the elements.

2. The times at which both side and soffit forms may be removed from a concrete element can conveniently be related to the strength achieved in the element.

ACKNOWLEDGMENTS

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ACCELERATED METHODS OF ESTIMATING THE STRENGTH OF CONCRETE

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This paper gives the results of an experimental investigation to develop equations for estimating the potential strength of concrete by using the accelerated-curing methods recommended by the American Society for Testing and Materials. Cylinders made from 21 different batches of concrete were tested. Ranges of variables included in this investigation were types of cements (1, 2, and K); types of molds (steel, plastic, and cardboard); water-cement ratios (by weight), 0.41 to 0.72; aggregate-cement ratios (by weight), 2.5 to 4.1; maximum size of coarse aggregate, 1 in. (25.4 mm); and the 28-day compressive strengths, 3,400 to 6,800 psi (23 500 to 46 900 kPa). An equation and correlation curves are presented for the locally used materials and mixes relating the strength of types 1, 2, and K cement concretes obtained in the accelerated-curing methods and the 28-day strength obtained with standard-curing conditions. Based on the results from this investigation and those of other investigators from various parts of the world, an equation applicable universally with reasonable accuracy is presented for estimating the potential strength of concrete by using accelerated-curing methods. Equations based on maturity concept are also presented to predict the strength of concrete when the water-cement ratio, duration, and temperature of curing are known. The results of this investigation are compared and analyzed in relation to published works.

•CONCRETE testing in the last 50 years has remained consistently the same; however, the methods of production and placement have changed quite considerably. Developments such as high-capacity ready-mix concrete plants, transit-mixer trucks, prefabricated forms, power vibrators and finishing equipment, conveyors, and concrete pumps have enabled contractors to place and finish huge quantities of concrete, as much as 120 to 400 yd^3 (92 to 306 m^3) in an hour.

What are the consequences? Buildings, roads, and bridges could be completed before one conventional 28-day, standard-cured concrete test cylinder can be tested. A large quantity of inferior concrete could be placed in a key location of a structure and become an integral part of the structure before it could be detected. It may then be too late to remove the inferior concrete alone; thus the removal of the entire structure may be needed. The problem therefore is to reduce the time required between the placement of concrete and the determination of the strength. This can be achieved by replacing the standard 28-day moist curing (ASTM C 684-71T) with accelerated curing. The accelerated curing reduces the waiting period from 28 days to 1 or 2 days (ASTM C 192-69). This alone is a sufficient and adequate reason for adopting accelerated-curing procedures.

Earlier research of accelerated curing has helped to develop three main requirements (2) for an acceptable test method: (a) The test must be reproducible, (b) the test must produce accelerated concrete strengths that can be easily related to a reference strength level (probably the 28-day standard-cured strength), and (c) the accelerated strength must be a high percentage of the reference strength to ensure that the test will be efficient. Recognizing the need for rapid test results, ASTM has adopted three accelerated-curing procedures (ASTM C 192-69): the warm-water curing

method, the boiling-water curing method, and the autogenous curing method.

The detailed accelerated-curing procedures are to be followed nationwide; however, the correlation curves and the relationships for predicting the 28-day compression strength of concrete will largely depend on the locally used materials and mixes.

In that type K expansive cement is the cement of the future (3), there is a need for developing relationships for predicting the 28-day strength of this shrinkage-compensating concrete by using accelerated-curing methods.

SPECIFIC AIMS

The specific aim of this research is to develop correlation curves for the locally used materials, mixes, and methods relating the strength of portland cement concrete obtained in the accelerated-curing methods and the 28-day strength obtained with standard-curing procedures.

The correlation between the compressive strength obtained from the accelerated method and from the 28-day standard-curing method will also be developed for the type K, expansive cement concrete.

EARLIER RESEARCH

Theoretical Basis

Concrete Hardening, The Curing Process

There are four distinct periods in the hardening or hydration process. The first period occurs when cement and water are brought into contact. It is a period of rapid dissolution and exothermic chemical reactions lasting about 5 min. The rate of reactions then subsides to a low level, and the second period, one of dormancy or plasticity, lasts 40 to 120 min (depending on the cement characteristics). The third period, one of rapid chemical reactions, then begins, usually lasting 3 to 6 hours, during which the concrete loses its plasticity and sets. Final set commonly occurs by the sixth hour after the cement first comes in contact with water. At that time approximately 85 percent of the hydration process is still remaining (2). After final set, the fourth period begins, one in which the chemical reactions continue at a diminishing rate until the conditions necessary for the reactions to continue are no longer present.

When hydration occurs at a rapid pace, there is rapid hardening and rapid gain of strength. Hydration, being an exothermal chemical reaction, gives off heat, and, if heat is applied externally, hydration will accelerate. When the rate of heat evolution is measured at normal temperature and at an elevated temperature (applying heat), it can be seen that the heat evolution at elevated temperatures is greater than at normal temperatures (4) (Figure 1). This implies that the hydration process is accelerated and that the strength gain due to elevated temperatures is also accelerated.

Maturity Concept

The compressive strength of concrete increases with time; the temperature at which it is cured also has a major influence. Previous research has shown the most accepted datum temperature to be between 10 and 14 F (-12.2 and -10 C). It is at this temperature and above that hydration and hardening begin. The maturity concept is based on this datum temperature, where

$$\text{Maturity (M)} = \text{curing time (hours)} \times \text{temperature of cure (deg F)} - 14 \qquad (1)$$

Figure 1. Rate of heat evolution versus time.

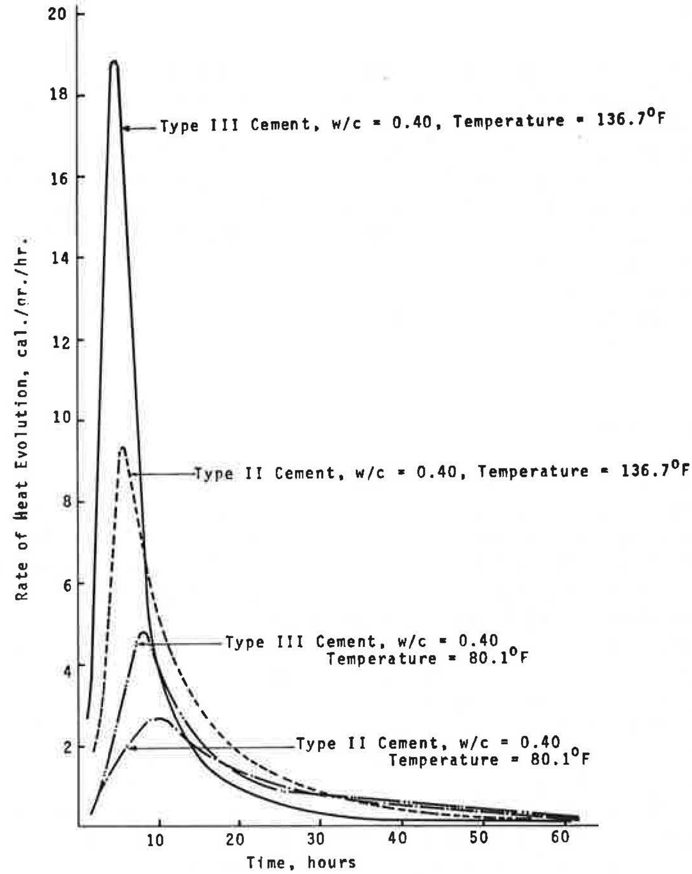
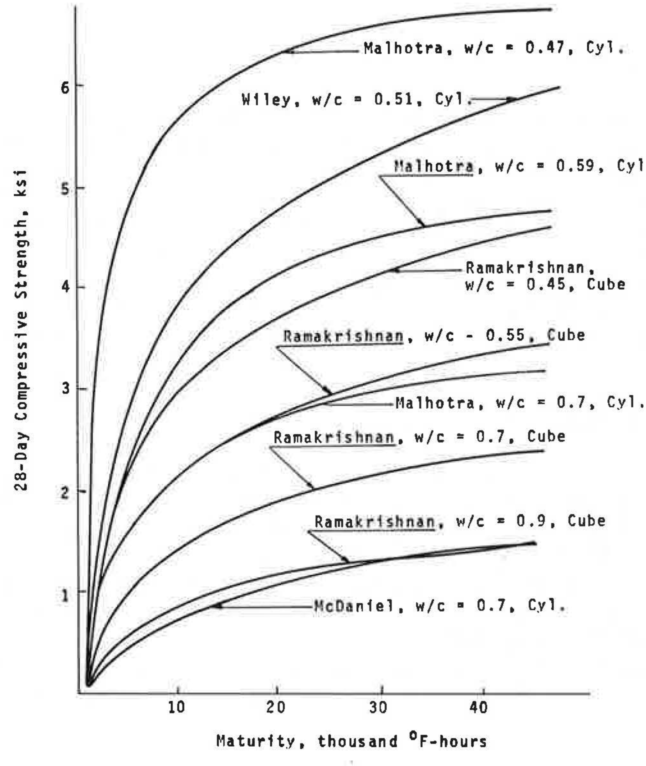


Figure 2. Twenty-eight-day strength versus maturity.



The units of maturity are deg F hours (deg C hours).

The strength then of a given concrete mix can be defined by a single value, maturity, all other conditions being equal. Figure 2 is based on the results of the research of McDaniel (6), Wiley (7), Malhotra and Berwanger (8), and Ramakrishnan and Li (9) and shows that the strength increase due to increasing maturity is not linear. If these same curves are plotted by using $\log_{10}$(maturity) as the independent variable on the X-axis, the curves become linear as shown in Figure 3. By using the maturity concept, it is possible to determine the strength of a given concrete without physically testing a concrete specimen.

Maturity, then, is an additional tool in determining the concrete strength, but the test data and results should not be used indiscriminately.

Findings

Previous research of boiling-water or high-temperature accelerated curing has shown a wide variety of procedures and methods. Results of research by McGhee (2), Malhotra and Berwanger (8), Smith and Chojnacki (11), Malhotra and Zoldners (12), and Vuorinen (13) are shown in Figure 4. The close grouping of these curves is quite extraordinary in spite of the fact that the testing was done in three countries and three different test methods were used. It also shows the possibility of using a general curve obtained from these curves to estimate the 28-day strength until a more accurate local curve can be developed. A local curve must be found because of differences in aggregate, sand, and cement in various regions.

The results of hot-water testing done by Malhotra (14) and McGhee (2) are shown in Figure 5. It can be seen from these curves that there is a wide range of results. It would not be possible to use an average equation to represent all the data; therefore separate equations based on specific cement types are definitely required. Figure 6 shows curves developed by McGhee (2) that compare efficiency (ratio of accelerated strength to 28-day strength) and the curing temperature. His testing and conclusions indicated that the highest efficiencies are obtained at water temperatures of 165 to 180 F (74 to 82 C).

SCOPE

This investigation consisted of two parts. In the first part the mandatory ASTM procedures for accelerated curing (ASTM C192-69) were followed. The second part consisted of minor variations of ASTM procedures necessary to time scheduling properly for testing and to improve the efficiency of the tests.

Included in the testing were 21 different batches of concrete obtained from local ready-mix plants. Ranges of variable parameters included in this investigation were water-cement ratios (by weight), 0.41 to 0.72; aggregate-cement ratios (by weight), 2.5 to 4.1; maximum size of coarse aggregate, 1 in. (25.4 mm); and the 28-day compressive strengths, 3,400 to 6,800 psi (23 500 to 46 900 kPa).

MATERIALS AND METHODS

The cements used were types 1, 2, and K portland cements. The fine aggregate used was a coarse, washed river sand found along the Cheyenne River near Hot Springs, South Dakota. The coarse aggregate used was crushed limestone from the local quarries of Rapid City, South Dakota. Concrete using type K cement was made in the laboratory. Concretes using types 1 and 2 cements were obtained from local ready-mix concrete producers, one using the central-mixer technique and the other using the transit-mixer operation.

The casting of the cylinders was done according to ASTM C192-69. Molds of steel, cardboard, and plastic were used, and both manual and mechanical methods of consoli-

Figure 3. Twenty-eight-day strength versus $\log_{10}$ (maturity).

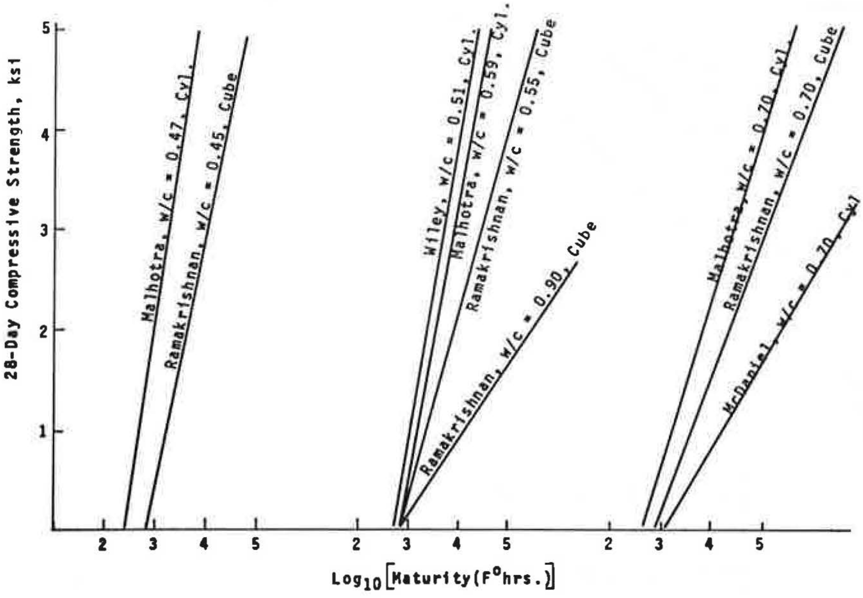


Figure 4. Twenty-eight-day versus accelerated strength at high temperatures.

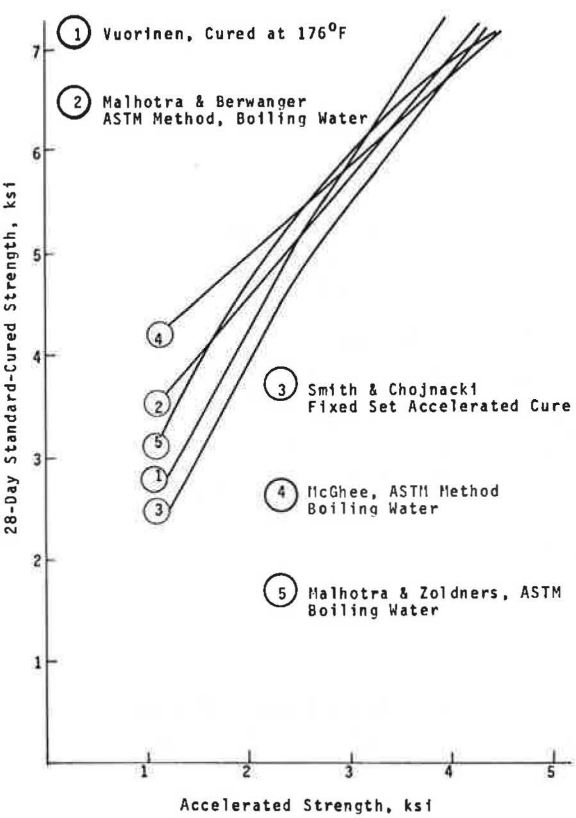


Figure 5. Twenty-eight-day versus accelerated strength at low and medium temperatures.

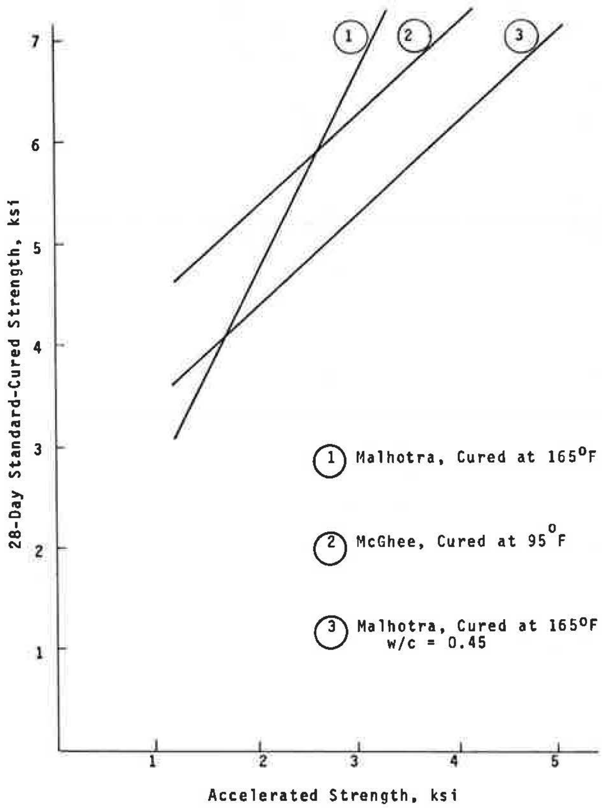
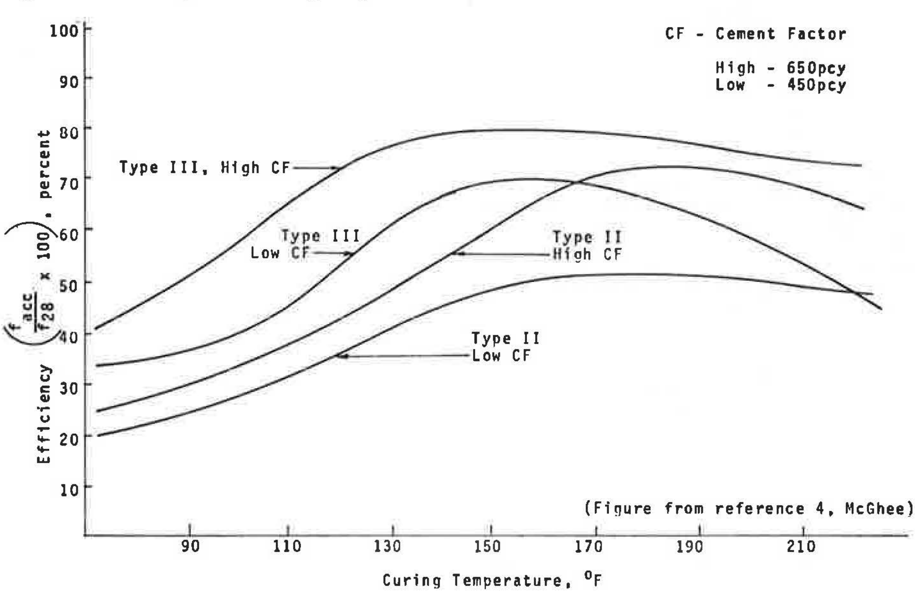


Figure 6. Efficiency versus curing temperature.



dation were used. A ½-in.-diameter (12.7-mm) rod and a 1¼-in. (31.75-mm) Stow concrete vibrator (model 71E) were used in consolidating the concrete.

The curing of the concrete cylinders was done according to ASTM procedures for standard curing (ASTM C192-69) and for accelerated curing (ASTM C684-71T). Additional accelerated-curing methods were also used, one of which was similar to the ASTM warm-water method. In this method, the curing water temperature was higher, 167 F (75 C), the duration of the cure was 1 hour longer, and the concrete was allowed to set for 2 hours before curing began. The other method was a slight variation of the ASTM boiling-water method. In this method, the cylinders were placed in the boiling water immediately after the cylinders from the hot-water test were removed. Because of this procedure, the time at which curing began in the boiling-water method was 26 hours after casting instead of the 23 hours recommended by ASTM. Therefore, the age at testing was also 3 hours later, at 31½ hours instead of 28½ hours. Table 1 gives the curing methods used and their variations.

Testing of the cylinder specimens was done according to ASTM C39-71 on a 300,000-lb (136 000-kg) capacity Tinius Olsen compression testing machine.

A minimum of nine cylinders were cast (all in the same type of mold) from each batch (three in warm or hot water, three in boiling water, and three for 28 days). Testing was also done on cylinders cast in different types of molds and from the same batch. The data obtained from these tests were used in determining regression equations.

DISCUSSION OF TEST RESULTS

The data obtained from the various test batches are given in Table 2.

Hot-Water Method

Tests 1 to 13 followed the hot-water procedure from Table 1. This method was recommended by an ASTM subcommittee and was effective July 1971. The resulting data are shown in Figure 7. By using the method of least squares (17) in determining regression equations, three equations representing different parameters were obtained and are also given in Table 3.

In the hot-water method, when all the available results with different variables are considered together, there are enough test results for a reasonable statistical analysis. There is a good correlation between the 28-day strength and the accelerated strength of concrete when the hot-water method is used. The efficiency $f_{acc}/f_{28'}$ (ratio of the accelerated strength to the 28-day strength) is greater when the hot-water method is used than when the other two methods are used. The efficiencies obtained for the hot-water, warm-water, and boiling-water methods are 51, 33, and 41 percent respectively.

Warm-Water Method

Tests 15 to 21 followed the warm-water procedure (Table 1) recommended by ASTM C684-71T. Test results are shown in Figure 8, and corresponding regression equations are given in Table 3. From Table 3 and Figure 8, one can see that the different types of cements are more sensitive to this type of cure than to the different types of molds used. This is shown by the low correlation factors obtained when results based on the use of different molds are individually analyzed. In the warm-water method, even at a comparatively low temperature of 95 F (35 C), there was a definite increase in the strength. There was also a consistent pattern in the rate of increase of strength of concrete, even though a small number of tests and a small range of strengths were used.

Among all the correlation coefficients found, the highest and lowest values occurred when this test method was used. If this method is to be used, it is imperative that the

Table 1. Curing methods used.

Procedure	Molds	Curing Medium	Curing Temperature (C)	Age Curing Begins	Duration of Cure	Age at Testing
ASTM 28-day standard	Reusable or single-use	Water	21	24 hours after casting	28 days	29 days
Hot-water	Reusable or single-use	Water	75	2 hours after casting	24 ± ½ hours	28 ± ¼ hours
ASTM warm-water	Reusable or single-use	Water	35	Immediately after casting	23½ ± ½ hours	24 ± ¼ hours
Boiling-water	Reusable or single-use	Water	93[a]	26 hours after casting	3½ ± 1/12 hours	31½ ± ¼ hours
ASTM boiling-water	Reusable or single-use	Water	93[a]	23 hours after casting	3½ ± 1/12 hours	28½ ± ¼ hours

[a]Test elevation = 3,230 ft (985 m).

Table 2. Test data.

Test No.	Type of Cement	Avg Strength Results (psi) — Steel Molds H-W	Steel B-W	Steel 28-D	Plastic Molds H-W	Plastic B-W	Plastic 28-D	Paper Molds H-W	Paper B-W	Paper 28-D
1	1	2,118	1,681	3,657						
2	1	2,192	1,630	3,498						
3	1	1,620	1,691	4,233						
4	1	1,630	1,750	4,292						
5	1		1,375	3,545		1,435	3,637			
6	1	3,682		5,237	3,554		4,977			
7	K	4,424	2,980	6,783		2,642				
8	1	1,896	1,753	3,896						
9	1	1,940	1,970	4,565						
10	K	4,346	2,567	6,443						
11	1	1,655	1,482	4,197						
12	1		1,687	3,836					1,599	3,538
13	1	1,939		4,244				1,189		4,069
14	1								2,167	4,721
15	1	1,416	1,802	4,234				1,308	1,611	4,054
16	K	1,198	1,747	4,900						4,638
17	1	1,490	1,610	4,200						
18	2	1,323	1,984	5,662		2,099	6,054			
19	K		2,120	5,860		2,064	5,736			
20	1	2,307		4,853	2,091		4,798	2,004		4,636
21	K	1,652	2,617	5,497				1,544	2,424	5,048

Note: H-W = hot-water, B-W = boiling-water, and 28-D = 28-day. 1 psi = 6.9 kPa.

Figure 7. Twenty-eight-day versus accelerated-strength results from the hot-water method.

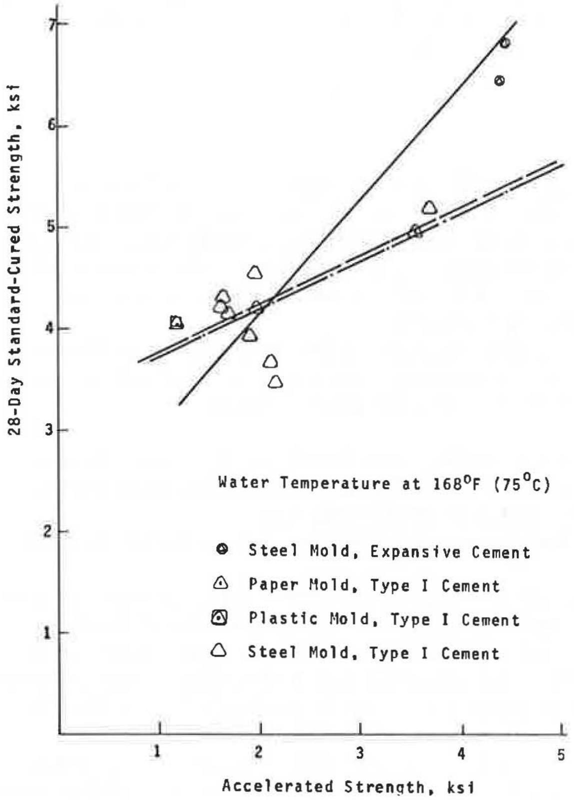


Table 3. Correlation equations.

Method	Test Parameters: Cement	Molds	No. of Tests	Equations	Correlation Coefficient
Hot-water	1	Steel	9	$f_{28} = 0.46\ f_{acc} + 3,248$	0.577
	1	All	11	$f_{28} = 0.458\ f_{acc} + 3,314$	0.592
	All	All	13	$f_{28} = 1.149\ f_{acc} + 1,840$	0.874
Warm-water	All	Steel	6	$f_{28} = -0.0075\ f_{acc} + 4,903$	-0.013
	All	All	10	$f_{28} = 0.095\ f_{acc} + 4,634$	0.059
	All	Paper	3	$f_{28} = 0.581\ f_{acc} + 3,639$	0.425
	K	All	3	$f_{28} = 1.053\ f_{acc} + 3,605$	0.838
	1	All	6	$f_{28} = 0.822\ f_{acc} + 3,008$	0.986
	1	Steel	3	$f_{28} = 0.729\ f_{acc} + 3,162$	0.992
Boiling-water	1	Steel	11	$f_{28} = 2.294\ f_{acc} + 169$	0.580
	K	All	8	$f_{28} = 1.158\ f_{acc} + 3,021$	0.703
	1	All	15	$f_{28} = 1.404\ f_{acc} + 1,644$	0.735
	K	Steel	5	$f_{28} = 0.258\ f_{acc} + 5,277$	0.832
	All	All	25	$f_{28} = 2.086\ f_{acc} + 682$	0.871
	All	Plastic	4	$f_{28} = 2.088\ f_{acc} + 1,077$	0.879
	1	Paper	3	$f_{28} = 4.053\ f_{acc} - 3,160$	0.907
	All	Steel	17	$f_{28} = 2.087\ f_{acc} + 681$	0.892
	All	Paper	4	$f_{28} = 1.571\ f_{acc} + 1,276$	0.954

various parameters involved in the casting and curing be separately analyzed; otherwise, the results of the regression equations will not be valid.

It also appears that the efficiency of a test is not an important consideration because the efficiencies found in this test were the lowest at 33 percent. Further research is needed to confirm the above-mentioned conclusions.

Boiling-Water Method

Two different boiling-water methods given in Table 1 were used. One method was recommended by ASTM. In the other, curing begins 3 hours later than in the correct ASTM procedure. Due to the close proximity of these methods, no differentiation will be made between them and both are called the boiling-water method. As shown in Figure 9 and given in Table 3, this 3-hour variation in initial curing before boiling appears to have little effect on the outcome of the results.

The correlations obtained by using the boiling-water method are similar to those of the results from the hot-water method when the same procedure was used for the analyses. However, the boiling-water method is easier to conduct because

1. There is no soaking of molds in curing water, and hence there is less cleanup;
2. It is easier to conduct the procedure in the field since no sophisticated curing tank is needed and any tank to hold boiling water is sufficient; and
3. It is possible to transport specimens for a short distance before curing beings.

The boiling-water method can be used when a variety of testing parameters and conditions are encountered on a project. If only one type of mold is used throughout a project but different cements are used, good correlation can be obtained (Table 3) by using the boiling-water method. It appears that different types of molds have a greater effect on the results than different types of cements. This is probably due to the 24-hour time interval between casting and the starting of the accelerated curing of the specimen. It allows the concrete enough time to reach the fourth stage of hydration, when the rate of release of the heat of hydration is decreasing (Figure 1). At this time, heat is supplied to the concrete by the boiling water, and the rate of strength development is accelerated.

A universal equation, such as the one shown in Figure 10, could be used for predicting the 28-day strength when initial data are sparse (new project, new area) or when cement types and molds are changing in a project. However, as Figure 9 shows, when one type of cement is used throughout the project, an equation representing the actual data will give a more accurate estimate of the 28-day strength. Figure 10 shows a single curve and equation that can be used when the boiling-water method is used. The lighter lines in Figure 10 are plots of equations from other investigators' work obtained from various parts of the world (2, 8, 11, 12, 13, 14).

On the whole, this method gives good correlations when compared to 28-day strengths and is quite easy to use. However, one disadvantage in this method is the overtime work necessary if the cylinders are cast after 1 p.m. If the cylinders were cast at 5 p.m., they would have to be placed in the boiling water at 4 p.m. the following day, removed at 7:30 p.m., and tested at 9:30 p.m. However, this overtime work is more than compensated for because of the ease in testing and the good correlations resulting even under adverse conditions. There is no overtime work involved in the use of the warm-water method if it is used during working hours.

Expansive Cement

Data obtained from testing concrete cylinders made of type K cement are given in Table 2. In the hot-water method, concrete made of type K cement developed higher efficiencies (66 percent) than concrete made of type 1 cement (51 percent). In the warm-water method concrete made of type K cement has an average efficiency of 27

Figure 8. Twenty-eight-day versus accelerated-strength results from the warm-water method.

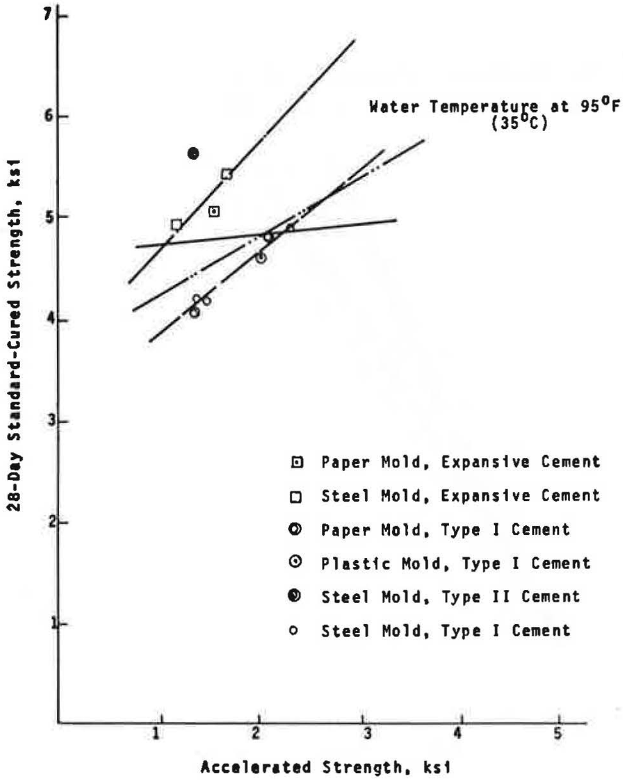


Figure 9. Twenty-eight-day versus accelerated-strength results from two procedures for boiling-water method.

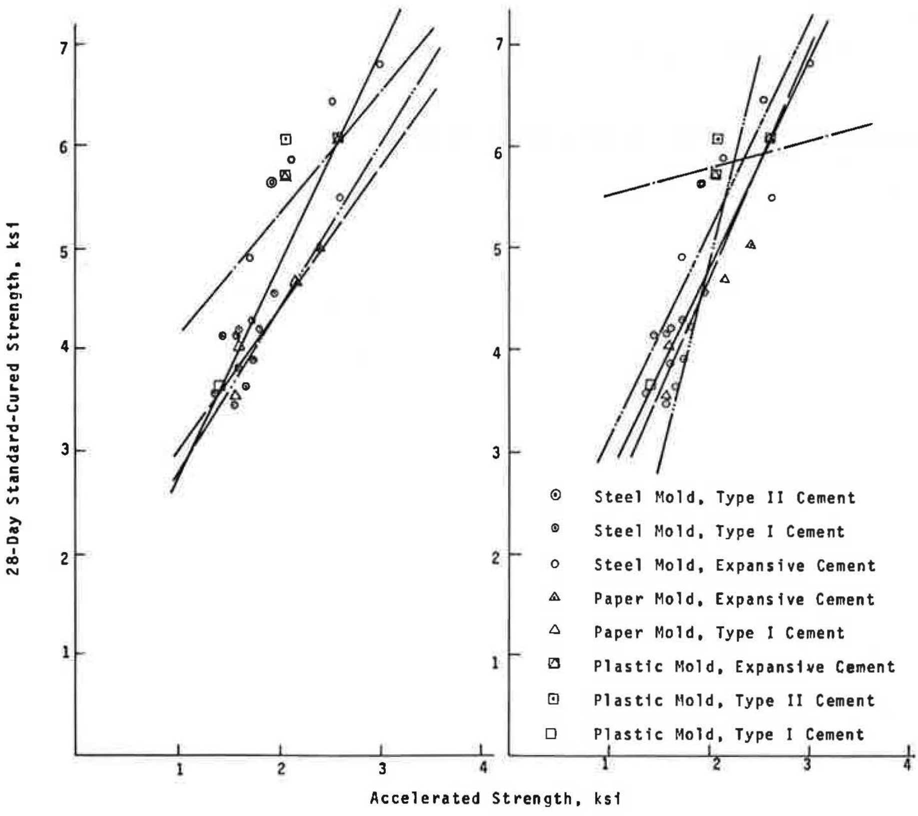


Figure 10. Twenty-eight-day versus accelerated-strength results from two equations for boiling-water method.

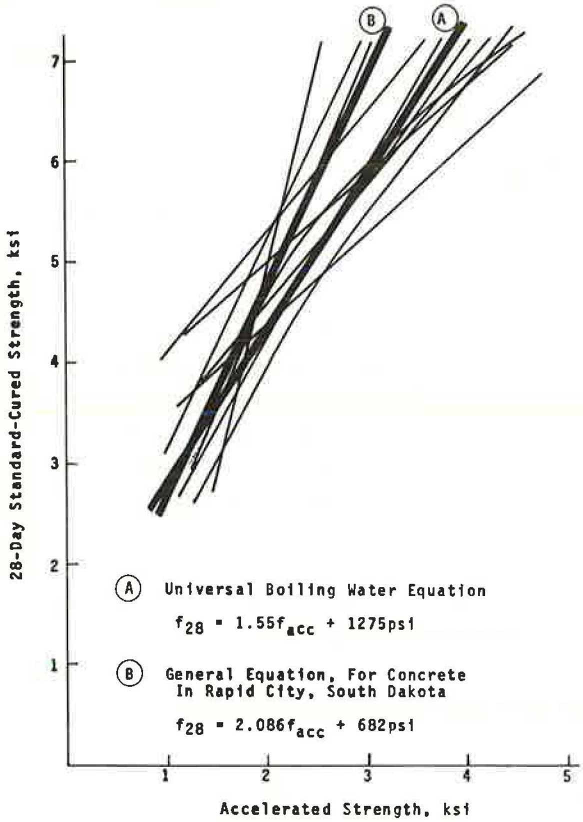
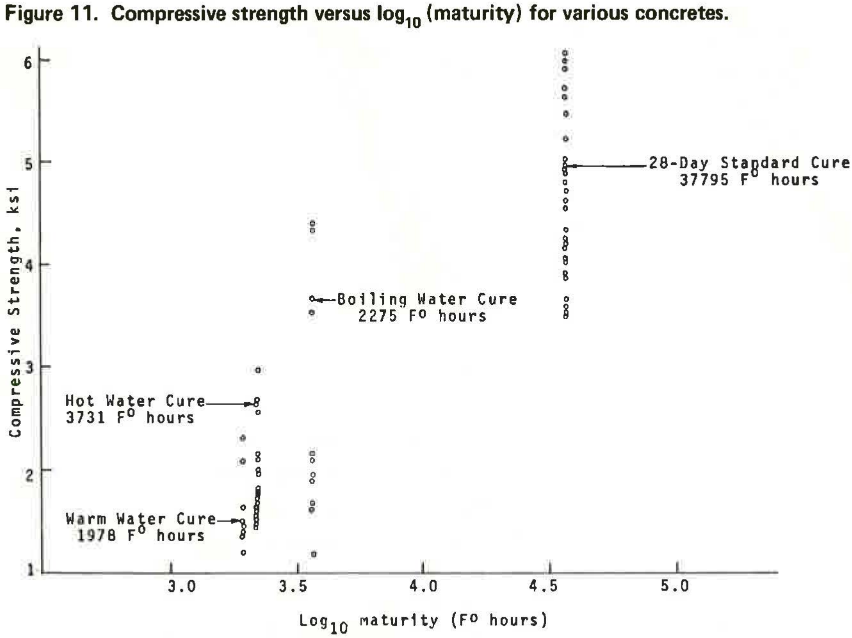


Figure 11. Compressive strength versus $\log_{10}$ (maturity) for various concretes.



percent versus 39 percent for type 1 cement concrete. In the boiling-water method, type K cement concrete had an efficiency of 41 percent, which is about the same as that of type 1 cement concrete (42 percent). In both hot-water and warm-water methods, the test points were few; it is therefore difficult to make specific comments about the results. Expansive cement concrete behaves differently in the hot-water and warm-water curing methods than type 1 cement concrete. Expansive cement concrete develops higher efficiencies in the hot-water method and lower efficiencies in the warm-water method than type 1 cement concrete. This shows that the rates of strength development are different for types 1 and K cement concretes. These two accelerated-curing methods (using hot and warm water) appear to accentuate the already inherent differences between types 1 and K cements that cause the cements to react differently.

When the boiling-water method is used, the data do not give a clear picture of any specific trends. The trend appears to be similar to type 1 cement concrete; however, more data points are needed to actually develop an equation that will estimate the 28-day strength of expansive cement concrete. At present the general equation (Figure 10) representing all cements and all molds for the area of Rapid City, South Dakota, appears to be able to estimate the 28-day strength of concrete made of types 1 and K cement with reasonable accuracy.

The 28-day strengths obtained by using type K cement range from 4,600 to 6,780 psi (31 700 to 46 780 kPa) with water-cement ratios ranging from 0.45 to 0.56. Further research is needed in regard to the accelerated curing of expansive cement concrete.

Maturity Method

The compressive strengths obtained for the various concretes are plotted against their $\log_{10}$ (maturity) values in Figure 11, which shows that there is no linear relation between compressive strength and $\log_{10}$ (maturity). Some researchers (6, 7, 8, 9) have recognized that the accuracy of the prediction of concrete strength based on the maturity concept can be improved by including water-cement ratio as one of the parameters in addition to the duration and temperature of curing. Ramakrishnan and Li (9) have proposed equations, based on maturity concept, in which water-cement ratio, duration and temperature of curing, and type of curing are the variables. The results from this investigation are compared with the proposed equations (9) in Figure 12. These equations show a reasonable agreement with the test results.

When the water-cement ratio is taken into consideration as shown in Figure 12, the test results show a better correlation with the predicted strengths. Table 4 gives equations for a certain water-cement ratio range that can be used to estimate the 28-day compressive strength. The accuracy of this method appears to be as good as that for the accelerated methods. In this method (Figure 12), differences in molds, cements, and curing have a pronounced effect on the results. As an example, cylinders cast in cardboard molds and cured for 28 days (standard cure) will have the same maturity but lower strength (approximately 10 to 15 percent lower) than cylinders cast in steel molds and cured 28 days (standard cure). Any equation derived to estimate the 28-day compressive strength or strength at any other age must take into consideration all influencing parameters that affect the strength results.

Effect of Accelerated Curing

The major effect of accelerated curing is accelerating the strength development of concrete. This can be done at various rates, depending on the type of cure used.

An adverse effect that has not been adequately recognized in earlier research is the fact that concrete cured by accelerated methods for a short period and then cured normally for the remaining period of 28 days in many cases will not develop the same compressive strength as that of a similar cylinder cured by standard methods for 28 days.

A few pilot tests were conducted to examine the effects of accelerated curing on the

Figure 12. Compressive strength versus $\log_{10}$ (maturity) for various water-cement ratios.

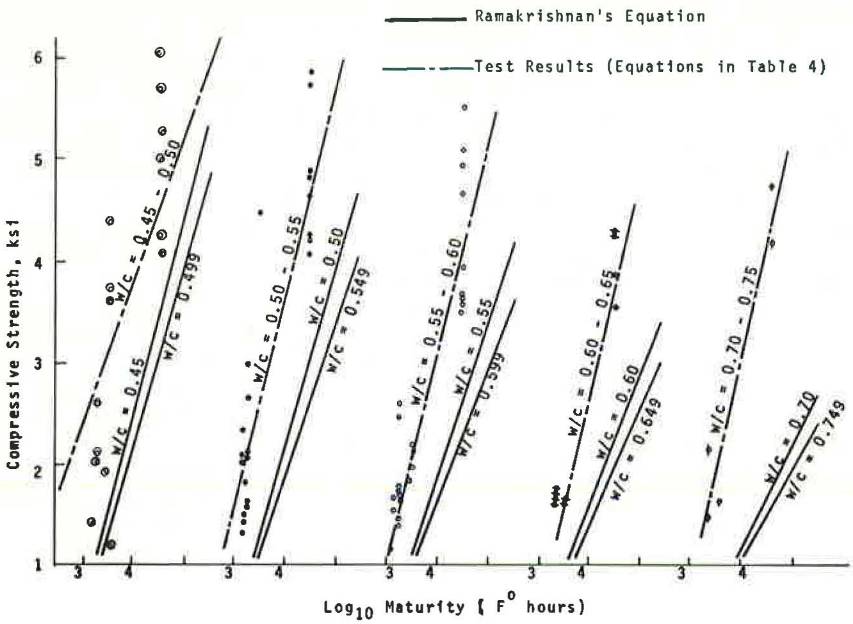


Table 4. Strength estimation equations based on maturity and various water-cement ratio ranges.

Cement	Mold	No. of Tests	Water-Cement Ratio Range	Equations	Correlation Coefficient
1, K	Steel Plastic Paper	16	0.45 to 0.50	f_c = 1,431 $\log_{10}$(maturity) - 1,949 psi	0.844
1, K	Steel Plastic	23	0.50 to 0.55	f_c = 2,033 $\log_{10}$(maturity) - 4,521 psi	0.889
1, K	Steel Plastic Paper	23	0.55 to 0.60	f_c = 2,065 $\log_{10}$(maturity) - 5,194 psi	0.819
1	Steel Paper	10	0.60 to 0.65	f_c = 1,969 $\log_{10}$(maturity) - 5,068 psi	0.880
			0.65 to 0.70	No test points in this range	
1	Steel Paper	5	0.70 to 0.75	f_c = 2,284 $\log_{10}$(maturity) - 6,035 psi	0.962

Note: 1 psi = 6.9 kPa.

Table 5. Effects of ASTM accelerated curing on 28-day compressive strength.

Test Batch	Curing Method	Water-Cement Ratio	Molds	No. of Tests	Cement Type	1-Day Strength (psi)		28-Day Strength (psi)	
						Accelerated	Normal	Accelerated	Normal
6	Hot-water at 168 F	0.48	Steel	1	1	3,682	800	4,061	5,237
18	Warm-water at 95 F	0.45	Steel	2	2	1,323	900	5,286	5,662
5	Boiling-water at 199 F	0.56	Steel	2	1	1,375	500	2,732	3,545
14	Boiling-water at 199 F	0.71	Paper	3	1	2,167	400	4,245	4,721
16	Boiling-water at 199 F	0.56	Steel	2	K	1,747	700	4,948	4,900
17	Boiling-water at 199 F	0.52	Steel	2	1	1,610	650	2,887	4,200

Note: 1 psi = 6.9 kPa. 1 F = 1.8 (C) + 32.

28-day compressive strength. These results are given in Table 5 (19). It appears that at higher curing temperatures the effects of accelerated curing are more pronounced. The best time to begin curing appears to be 4 to 6 hours after casting; the lower curing temperatures will have less long-term effect on concrete compressive strength (19, 20).

When accelerated methods are used in strength estimation, the previously mentioned adverse effect need not be considered because it does not affect the results. If, however, accelerated methods are used for other purposes, such as achieving high early strength in concrete by steam curing, high-temperature-immersion curing, or other methods, this long-term effect should be considered.

CONCLUSIONS

1. The boiling-water method can be used when a variety of testing parameters and conditions are encountered on a project. Currently, we recommend its use because of the ease in testing, its satisfactory correlations with standard-curing methods, and the shortage of information regarding the other accelerated methods.
2. There is need for more research work using the hot- and warm-water methods over a greater range of strengths before any one method can be recommended. This could be done in conjunction with work that will expand the maturity concept with actual measurements of temperatures inside the cylinders.
3. Expansive cement concrete reacts favorably to accelerated curing. Because of its inherent physical and chemical properties, expansive cement concrete has strength development rates that are different from those of type 1 cement concrete. When expansive cement concrete becomes more widely accepted and used, separate correlation curves should be developed.
4. Accelerated methods are needed and can benefit the owner, construction contractor, and concrete producer in the form of reduced costs, less chance of construction delay due to testing uncertainty, and more uniform concrete throughout the project.
5. When the advantage of using accelerated testing is understood and accepted by all engineers, contractors, and testing agencies, this could possibly be the only test used as a measure of the strength of concrete.
6. Casting in steel molds resulted in concrete cylinder strengths 10 to 15 percent higher than in paper molds and 3 to 6 percent higher than in plastic molds. These conclusions were reached by comparing concrete strengths obtained when different molds were used in the same test batch.

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PRACTICAL APPLICATION OF THE MATURITY CONCEPT TO DETERMINE IN SITU STRENGTH OF CONCRETE

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The C. N. Tower will be the highest free-standing structure in the world. The foundations contain 10,000 yd^3 (7646 m^3) of low-heat cement concrete, and accelerated testing was used throughout the concreting period. The slip-formed concrete superstructure is 1,464 ft (450 m) high and contains 40,000 yd^3 (30 580 m^3) of concrete. The accelerated test used during slip forming was the 48-hour autogenous test devised by Smith and Tiede. The last 3 months of slip forming were carried out from December 1973 to February 1974, the coldest period of the Canadian winter. Ambient conditions as low as 1 F (-17 C) were experienced, and wind-chill factors at heights above 1,000 ft (305 m) significantly increased the low-temperature effects. Maturity testing was done to check that, as the slip form rose, the concrete exposed to winter conditions achieved adequate strength. Thermocouples inserted during every shift were monitored at intervals throughout each 24-hour period, and in-place strengths were calculated. Monitoring continued until acceptable strengths had been achieved at each level. Evaluation was based on cores drilled from the structure and cylinder tests.

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The last 3 months of slip forming were carried out from December 1973 to February 1974, the coldest period of the Canadian winter. Ambient conditions as low as 1 F (-17 C) were experienced during this period, and wind-chill factors at heights above 1,000 ft (305 m) significantly increased the low-temperature effects.

Maturity testing was carried out to check that, as the slip form rose, the concrete exposed to winter conditions achieved adequate strength. Thermocouples inserted during every shift were monitored at intervals throughout each 24-hour period, and in-place strengths were immediately calculated. Monitoring continued until an acceptable strength had been achieved at each level. Evaluation was based on cores drilled from the structure as well as cylinder tests.

COLD WEATHER PROBLEMS

Problems presented by cold weather are threefold. First, the steady rise of the slip form depends on the setting of the concrete proceeding normally during the few hours the concrete is in the form. Second, the concrete is only in the form for a few hours before it is exposed to the weather. Third, should the concrete on emerging into the cold ambient conditions be cooled before it has achieved adequate strength, the continued rise of the slip form could result in a significant height of the C. N. Tower above

a weak layer or layers of concrete.

The initial setting of the concrete for the C. N. Tower in Canada was fairly simply controlled by the use of heated concrete and radiant heaters directed at the slip form at the finishing level.

DESIGN OF PROTECTION

Protection of the concrete in the C. N. Tower after it had emerged from the form was a little more complex. Three-dimensional calculations showed that without protection the concrete would cool rapidly to ambient temperatures before it had gained adequate strength (Figure 1). Two to 3 days of protection with 1 in. (25.4 mm) of polystyrene would however give adequate strength. At a planned maximum rise of 12 ft/day (3.65 m), this meant that about 36 ft (10.97 m) of protection would hang below the slip form. However, the temperature gradients resulting as the concrete emerged into the cold air could crack the concrete (Figure 2). The final configuration of the protection was therefore 20 ft (6.09 m) of 1-in. (25.4-mm) polystyrene, 9 ft (2.74 m) of ½-in. (12.7-mm)polystyrene, and 8 ft (2.43 m) of ¼-in. (6.4-mm) plywood. This combination provided adequate protection for initial strength gain. It also allowed the concrete to cool sufficiently before it was exposed to the atmosphere to avoid temperature gradients severe enough to cause cracking (Figure 3).

STANDARD AND ACCELERATED TESTING

Throughout the concreting program, sets of five test cylinders were cast for each 75 yd^3 (57 m^3) of concrete.

Two cylinders were subjected to accelerated testing after 48 hours by use of the autogenous method developed by Smith and Tiede. The remaining cylinders were given standard laboratory fog-room curing. One was tested at 7 days, two were tested at 28 days, and one was tested at 90 days. Analysis of all standard cylinder tests gave the following results [the superstructure is 620 lb (280 kg) of varying proportions of types 1 and 4 cement per cubic yard (cubic meter) (1 psi = 6.9 kPa)]:

Age at Test (Days)	Mean Strength (psi)	Standard Deviation (psi)	Coefficient of Variation
7	5,190	432	8.3
28	6,900	432	6.1

Typical data devised from regression analysis for the relationships of accelerated tests to standard 28-day cylinder tests were as follows (1 psi = 6.9 kPa):

Cement Type	No. of Tests	y = mx + b	Sy·x (psi)	Correlation Coefficient	95 Percent Level of Signif-icance
1	43	y(28-day) = +1.3027 Ra(48-hour) + 1,392	341	0.8227	Yes
4	147	y(28-day) = 0.9162 Ra(48-hour) + 3,611	356	0.7173	Yes

Figure 1. Temperature curves for walls; no protection below finishing platform.

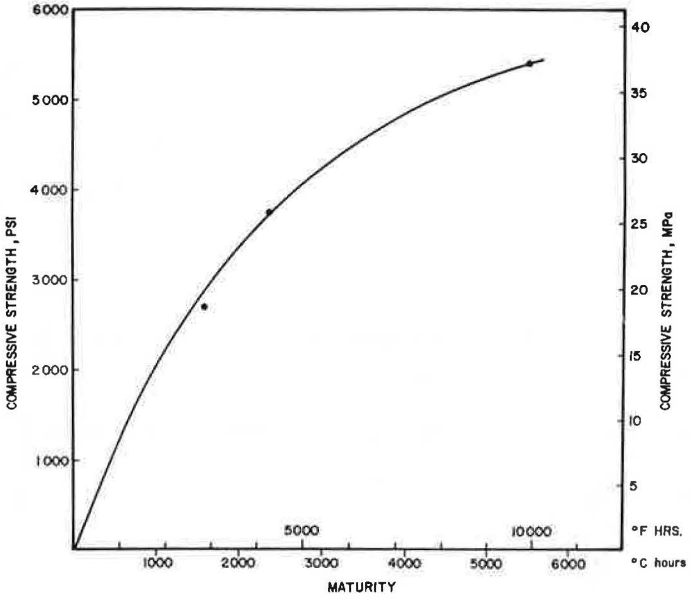
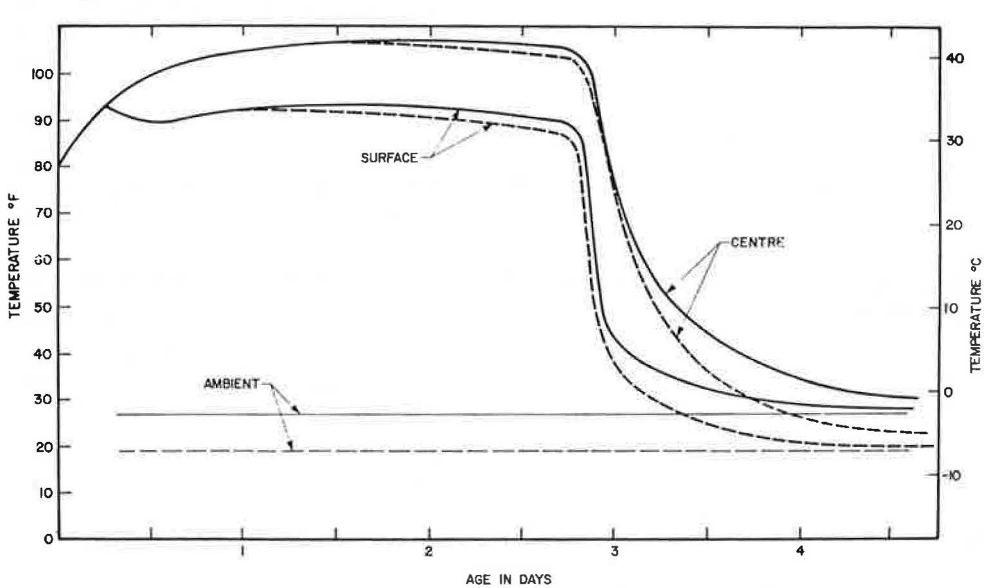


Figure 2. Temperature curves for walls; 36 ft (10.97 m) of 1-in. (25.4-mm) polystyrene below finishing platform.



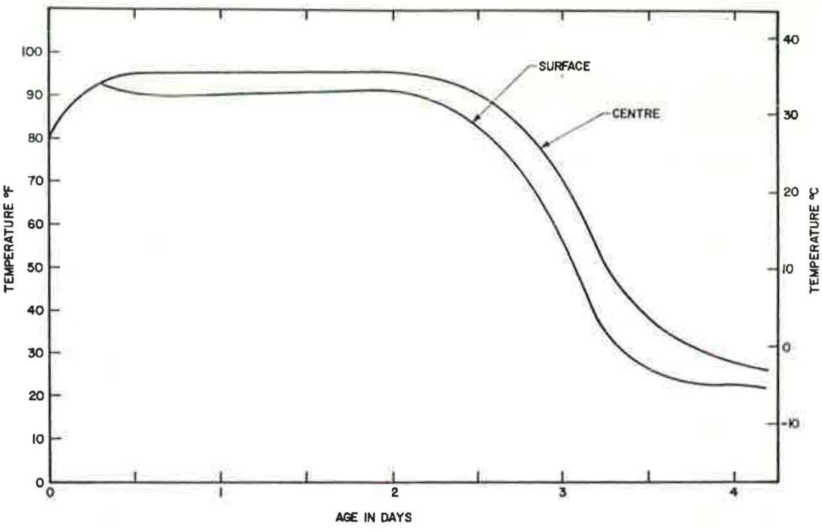


Figure 3. Temperature curves for walls; 20 ft (6.09 m) of 1-in. (25.4-mm) polystyrene, 9 ft (2.74 m) of ½-in. (12.7-mm) polystyrene, and 8 ft (2.43 m) of ¼-in. (6.4 mm) plywood below finishing platform.

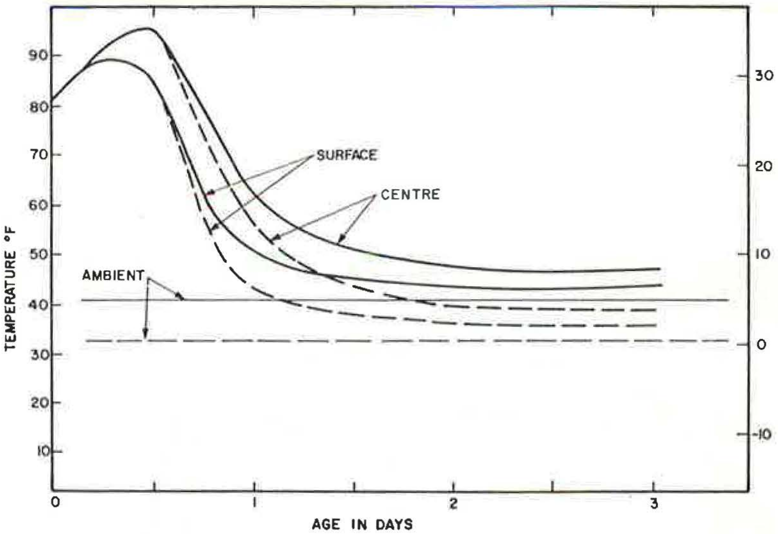


Figure 4. Strength-maturity plots for three thermocouples.

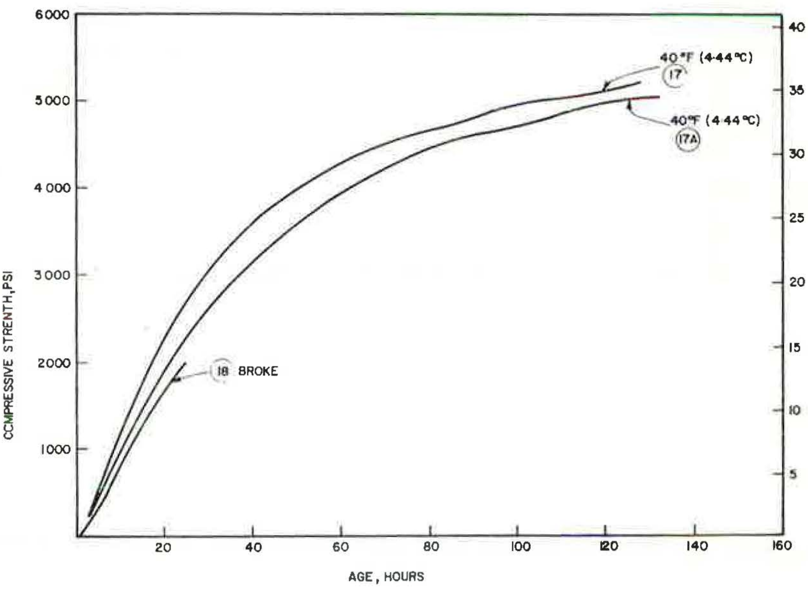


Figure 5. Strength-maturity curves for concrete mix used.

The concrete was uniform, and correlation coefficients for prediction of the 28-day strength from accelerated tests were good.

MATURITY TESTING

Before continual maturity testing was started on the C. N. Tower, some preliminary tests were made. Two cores drilled from the section of the structure chosen for the experiment were both tested at 4,710 psi (32.5 MPa) and at 7 days. Corrected to allow for the effect of overnight laboratory storage, these cores indicated in-place strength of 4,400 psi (30.3 MPa). In-place strength predicted from thermocouple readings of the same concrete was 4,250 psi (29.3 MPa).

After it was determined what strengths could be achieved, field checks were necessary to ensure they were consistently achieved. During each shift, a number of thermocouples were installed in the walls. Each thermocouple was on a free-turning reel fixed to the formwork to support the framework. As the form rose, the thermocouple wire unwound into the rising concrete. Readings were taken a number of times a day and were plotted on a standard form (Figure 4). Strength values were based on a strength-maturity curve previously established for the mix in use (Figure 5). The in-place strength of each shift of concrete was therefore known each day as it developed.

CONCLUSIONS

By the use of accelerated testing and in-place strength determinations, the adequacy of the concrete and thus the safety of the structure were continuously monitored. The results of the potential and actual strengths were available during the first 3 days after the concrete was placed. The concrete in the C. N. Tower was therefore known to be strong enough before it emerged from its protection.

Developments of this combination of advanced testing techniques offer great potential to the industry for safety and economy.

FIELD EVALUATION OF EXPANDED POLYSTYRENE MOLDS FOR SELF-CURED, ACCELERATED STRENGTH TESTING OF CONCRETE

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An accelerated-curing method that offers the possibility of forecasting the 28-day strength of concrete 48 hours after sampling was evaluated in the field. The method consists of casting and curing the concrete in styrofoam molds and has the following advantages over conventional accelerated-curing methods: (a) There is no change in the sampling and testing procedures normally used for the standard 28-day test, (b) the test is done during normal working hours, (c) no special heating or curing equipment is needed, (d) the concrete sample is protected from large curing-temperature variations that may occur during handling and transportation from the field to the laboratory, (e) the test cylinder of green concrete is protected from rough-handling damage, and (f) the technician is not exposed to injury by heat. More than 1,300 cylinders of paving and structural concrete, produced by 4 suppliers, were tested. The study showed that (a) there is a good correlation between 48-hour and 28-day test results, (b) the gain in strength at 48 hours is more than 60 percent of the 28-day strength, (c) a different 48-hour to 28-day strength regression equation was found for each concrete supplier, and (d) the reliability of the strength test results of cured samples obtained by using the expanded polystyrene mold method is of the same order as that obtained by more elaborate accelerated-curing methods.

•THE advantage of early determination of concrete strength potential has been recognized for many years. Accelerated strength tests for concrete have been used since 1920, and a considerable amount of work has been done in the 1960s (1, 2, 3, 4). In 1963, the Canadian Standards Association (CSA) and the American Society for Testing and Materials (ASTM) appointed subcommittees to study the development of accelerated strength tests for concrete. It was not until the beginning of 1970 that CSA and ASTM adopted tentative methods for making, curing, and determining the compressive strength of accelerated-cured concrete test specimens (CSA A 23.2-1973 and ASTM C 684-73T).

CSA A 23.2-1973 offers a modified boiling-water method and an autogenous curing method (CSA A 23.2.26). ASTM C 684-73T also included the warm-water method. A brief description of the three accelerated-curing procedures is given in Table 1.

Although it is evident that an early strength test, in lieu of the 28-day test, is an effective tool to improve quality control and promote early decision making, none of the standardized procedures have come into general use. The proposed methods require special equipment and curing procedures, they complicate testing, they are awkward to perform, and they increase the cost of testing significantly.

SCOPE

The purpose of this study was to determine if a truly simple accelerated strength test

method could be achieved by using a single-use expanded polystyrene mold such as the one originally developed to protect concrete test cylinders from frost or handling damage (5). The inherent insulating property of the mold is used to accelerate the curing of the concrete so that the potential 28-day compressive strength can be predicted from the 48-hour self-accelerated strength.

The study is divided into two main parts:

1. Evaluation of the self-cured accelerated strength test results, and
2. Comparison between the results of the proposed method and other accelerated strength test methods.

The evaluation was based on testing of a large number of field-sampled specimens of ready-mixed concrete, for which the sampling technicians received no special instructions. The concrete was produced for paving and structural works by four suppliers. All concretes used portland cement concrete (CSA type 10 or ASTM type 1), limestone coarse aggregates, natural or manufactured sand, and air-entraining and water-reducing admixtures (CSA type WN or ASTM type A).

EQUIPMENT AND TEST PROCEDURES

Figure 1 shows the commercially available expanded polystyrene mold used in this experiment. It meets all CSA and ASTM requirements for single-use molds.

The main features of the mold pertinent to this study are as follows:

1. It is made of expanded polystyrene so that the heat generated by the exothermic chemical reaction during cement hydration can be partially retained within the concrete to accelerate the curing, and
2. It has tight press-fit cover to prevent heat and moisture losses.

Standard procedures are followed for molding the specimens, which are cured in their molds at room temperature [60 to 80 F (15.6 to 26.7 C)] at the job site or the central laboratory until testing. No special curing facilities are required; the insulation of the mold provides for the self-accelerated curing. Cylinders are demolded immediately before testing, and CSA or ASTM procedures are used for capping and load testing. The concrete specimens are tested at, or about, 48 hours. A correction factor may be applied for discrepancies in testing time. However, in this study, corrections based on the strength-time curve of laboratory mix concrete (see Appendix) have not indicated any substantial improvement in the accuracy of the method. A strength-time curve up to 96 hours based on the self-cured method should be established to eliminate weekend testing and to reduce testing expenses.

FIELD EXPERIMENT

Description

The experiment was carried out during construction of different contract sections of the Trans-Canada Highway through the city of Montreal, which includes elevated and depressed roadways and a unique multilevel underground interchange. The concrete, supplied by four producers, consisted of several classes of concrete with different strength or durability requirements based on water-cement ratios. All concrete samples were collected at the job site as a part of normal field control work. The temperature, slump, and air content of each concrete sample were obtained before molding. Five 6 by 12-in. (15 by 30-cm) cylinders were molded from each concrete sample by using standard methods. Expanded polystyrene molds were used for the two accelerated-

Table 1. Accelerated-curing procedures.

Procedure	Molds	Accelerated Curing Medium	Accelerated Curing Temperature (F)	Age Accelerated Curing Begins	Duration of Accelerated Curing (hours, min)	Age at Testing (hours, min)
Warm-water	Reusable or single-use	Water	95	Immediately after casting of cylinder	$23\frac{1}{2}$, ±30	24, ±15
Boiling-water	Reusable or single-use	Water	212	23 hours after casting	$3\frac{1}{2}$, ±5	$28\frac{1}{2}$, ±15
Autogenous	Single-use	Heat of hydration	Initial concrete temperature augmented by heat of hydration	Immediately after casting	48, ±15	49, ±15

Note: 1 F = 1.8 (C) + 32.

Figure 1. Commercially available expanded polystyrene mold used in test.

Table 2. Number of samples and cylinders tested for each contractor.

Supplier	No. of Samples	No. of Cylinders
A	100	500
B	35	175
C	30	150
D	48	240
Total	213	1,065

cured cylinders; standard paraffin-cardboard molds were used for one normal-cured, 7-day cylinder and two normal-cured, 28-day cylinders.

All concrete samples were kept at room temperature in the field laboratory for the first 20 to 30 hours before they were transported to the central laboratory for testing. The 7-day and 28-day cylinders were cured normally; the samples cast in the expanded polystyrene molds were demolded immediately before testing at 48 (±5) hours.

The slight discrepancies between the actual and the target testing time were accepted to avoid overtime and to accommodate laboratory procedures.

The number of samples and cylinders tested for each contractor is given in Table 2.

Analysis of Test Results

The statistical analysis of all test results obtained for each supplier is given in Table 3. The analysis includes all results from concretes of widely different strength levels, slump, and air content requirements.

A summary of the statistical analysis of strength test results of different classes of concrete from each supplier is given in Table 4. The results show that the concretes sampled in this investigation may be rated good to excellent.

The repeatability of results of the accelerated strength test and the normal 28-day compression test obtained with concrete from different suppliers is given in Table 5. The repeatability is expressed in terms of coefficient of variation, which is estimated from the mean strength of all cylinders and the mean range calculated from individual range values of companion cylinders.

From the test results, we can conclude that the within-test variation of the field control is good to excellent and that the proposed accelerated strengths have a lower standard deviation than the normal 28-day strengths; however, the coefficients of variation are approximately the same.

Correlation Between Accelerated and Normal 28-Day Strengths

A series of linear correlations have been made to find the relationships among the accelerated strength, the 7-day strength, and the 28-day strength.

Table 6 shows the best fit lines for the uncorrected-accelerated and 28-day strengths, the corrected-accelerated and 28-day strengths, and the 7-day and 28-day strengths for each of the four suppliers. The regression equations, standard errors of estimate, and the correlation coefficients are also given.

The uncorrected value is the strength obtained at time of test (48 hours ± 5), and the corrected value is the strength results obtained at the time of test other than 48 hours, corrected to represent exactly 48 hours. The corrections are based on the data in Table 7. In relation to the data in Tables 6 and 7, the following observations can be made:

1. A different equation was used for each concrete supplier.
2. The correlation coefficient and the standard error of estimate are both improved when the 7-day strength values are used to predict the 28-day strength.
3. In general, a supplier with a good coefficient of correlation between 7-day and 28-day strengths will also have a good coefficient of correlation between the accelerated and the 28-day strengths. This also appears valid for the standard error of estimate.
4. The corrected results have little influence on the nature or the error of correlation.

The percentage ratios of the accelerated and 28-day strengths and the 7-day and 28-day strengths (strength gained) obtained for concretes produced by different suppliers are given below.

Table 3. Statistical analysis of strength test results.

Supplier	Value	Compressive Strength (psi): Accelerated Curing: No Time Correction	Accelerated Curing: Time Correction	Normal Curing: 7-Day	Normal Curing: 28-Day	Slump (in.)	Air Content (percent)	Concrete Temperature (F)
A	Mean	2,910	2,910	3,900	4,745	3.31	5.47	69.3
	Minimum	1,310	1,370	2,120	2,615	1.25	1.50	58.0
	Maximum	4,425	4,410	5,300	6,065	8.00	8.00	78.0
	Range	3,115	3,040	3,180	3,450	6.75	6.50	20.0
	Standard deviation	596	581	571	608	1.00	1.30	3.5
B	Mean	3,505	3,530	4,245	5,055	3.12	5.70	72.4
	Minimum	2,795	2,780	3,710	4,385	2.50	4.40	60.0
	Maximum	4,370	4,430	4,920	5,625	4.25	7.30	82.0
	Range	1,575	1,650	1,210	1,240	1.75	2.90	22.0
	Standard deviation	355	370	282	339	0.48	0.75	5.7
C	Mean	3,045	3,075	3,920	4,740	3.54	5.54	70.2
	Minimum	1,890	1,950	2,550	3,080	2.00	2.40	52.0
	Maximum	3,920	4,000	4,880	5,605	4.50	7.90	80.0
	Range	2,030	2,050	2,330	2,525	2.50	5.50	28.0
	Standard deviation	405	400	430	476	0.59	1.14	7.18
D	Mean	2,780	2,780	3,625	4,400	3.17	4.72	68.9
	Minimum	1,630	1,620	2,510	3,415	1.75	1.00	60.0
	Maximum	3,505	3,565	4,420	5,440	7.00	7.80	80.0
	Range	1,875	1,945	1,910	2,025	5.25	6.20	20.0
	Standard deviation	505	506	483	495	0.79	1.35	4.9
All data	Mean	2,980	2,990	3,880	4,695	3.31	5.47	69.3
	Minimum	1,310	1,370	2,120	2,615	1.25	1.50	58.0
	Maximum	4,425	4,430	5,300	6,065	8.00	8.00	78.0
	Range	3,115	3,060	3,180	3,450	6.75	6.50	20.0
	Standard deviation	559	557	522	561	1.004	1.29	3.5

Note: 1 psi = 6.9 kPa. 1 in. = 2.5 cm. 1 F = 1.8 (C) + 32.

Table 4. Concrete quality.

Supplier	Water-Cement Ratio	Mean Strength, 28-Day (psi)	Standard Deviation (psi)	Coefficient of Variation (percent)	Rating of Concrete Control[a]
A	0.49	5,025	331	6.6	Excellent
A	0.53	4,900	407	8.3	Excellent
A	0.70	3,775	498	13.2	Good
B	0.49	5,020	328	6.5	Excellent
C	0.47	4,820	344	7.1	Excellent
D	0.47	4,525	264	5.8	Excellent
D	0.63	4,020	533	13.3	Good

Note: 1 psi = 6.9 kPa.

[a]ACI 214.

Table 5. Repeatability of results of accelerated and normal strength tests.

	Accelerated Curing			Normal 28-Day Curing		
Supplier	Mean Range (psi)	Standard Deviation (psi)	Coefficient of Variation (percent)	Mean Range (psi)	Standard Deviation (psi)	Coefficient of Variation (percent)
A	157	139	4.69	203	180	3.78
B	125	111	3.30	201	178	3.60
C	171	152	4.91	236	209	4.43
D	135	120	4.27	195	173	3.98
Avg	145	129	4.32	217	192	4.11

Note: 1 psi = 6.9 kPa.

Table 6. Relationships of uncorrected-accelerated and 28-day strengths, corrected-accelerated and 28-day strengths, and 7-day and 28-day strengths.

			Regression Equation			
Strength	Supplier	Number of Samples	Constant	Slope	Standard Error of Estimate (psi)	Correlation Coefficient
Uncorrected-accelerated and 28-day[a]	A	100	2,190	0.88	310	0.86
	B	35	3,040	0.57	275	0.60
	C	30	2,134	0.86	333	0.73
	D	48	2,405	0.72	339	0.73
	All data	213	2,284	0.80	333	0.80
Corrected-accelerated and 28-day[a]	A	100	2,118	0.90	309	0.86
	B	35	3,024	0.57	268	0.63
	C	30	2,238	0.81	355	0.68
	D	48	2,381	0.72	335	0.74
	All data	213	2,263	0.81	333	0.80
Seven- and 28-day[b]	A	100	1,027	0.95	272	0.90
	B	35	1,758	0.78	263	0.65
	C	30	1,145	0.92	272	0.83
	D	48	1,322	0.85	279	0.83
	All data	213	1,030	0.94	269	0.87

Note: 1 psi = 6.9 kPa.

[a]Regression equation for 28-day strength = constant + slope X (accelerated strength).

[b]Regression equation for 28-day strength = constant + slope X (7-day strength).

Table 7. Correction factors for discrepancies in testing time.

Age at Test (hours)	Correction Factor (psi)	Age at Test (hours)	Correction Factor (psi)
36	275	49	-17
37	250	50	-34
38	225	51	-50
39	200	52	-67
40	175	53	-84
41	150	54	-100
42	125	55	-112
43	100	56	-125
44	80	57	-137
45	60	58	-150
46	40	59	-167
47	20	60	-185
48	0		

Note: 1 psi = 6.9 kPa.

Supplier	Accelerated/28-Day	7-Day/28-Day
A	61	81
B	69	84
C	64	83
D	63	82
All data	63	83

The 7-day strengths slightly exceed 80 percent of the 28-day strengths, and the accelerated tests vary from 60 to 68 percent.

Figures 2 and 3 show the combined data for tests of accelerated versus 28-day strengths and 7-day versus 28-day strengths.

ACCELERATED STRENGTH TESTS VERSUS METHOD USING EXPANDED POLYSTYRENE MOLD

The results of field evaluation of accelerated strength tests reported by Smith and Tiede (2), Malhotra and Zoldners (3), and Rodway and Ward (4) are compared with those of the present investigation. The method used and the correlations obtained by different investigations are given in Table 8. Although the slopes and the constant of the regression equation are quite different, the correlation coefficients are all between 0.8 and 0.9, and the standard error of the correlation in all three cases is in the range of 300 to 350 psi (2068 to 2413 kPa). The slightly lower correlation coefficient obtained in this study (0.8) may be attributable to the narrow range in results of 28-day compressive tests of the concrete used to establish the relationship (i.e., maximum/minimum value).

The relationships between accelerated and 28-day tests, from different studies, are shown in Figure 4. Although there is a general agreement in the trend, the origins [at 1,000 psi (6895 kPa)] or the slopes of the lines are quite different. This is not surprising since the procedures of the accelerated-curing methods used are different and the cement and concrete characteristics vary with localities.

CONCLUSIONS

Expanded polystyrene molds, such as those used to protect concrete cylinders against frost and jarring damage, can be used for field evaluation of self-cured, accelerated strengths of concrete specimens in routine quality control of concrete. Under the conditions in this study, the field evaluation indicated that the method is simple, practical, and reliable because

1. It makes use of expanded polystyrene molds of normal dimensions and involves no special equipment and procedures,
2. The complete test can be done during normal working hours,
3. The repeatability of test is equal to or better than the presently accepted standard 28-day strength test, and
4. The reliability of this method compares favorably with that of other accelerated test methods in predicting the 28-day strength.

This study indicates that an accelerated testing procedure using expanded polystyrene molds shows considerable promise and that further study should be made to investigate the effects of special environmental conditions and concrete mixes.

Figure 2. Two-day uncorrected-accelerated versus 28-day strength results.

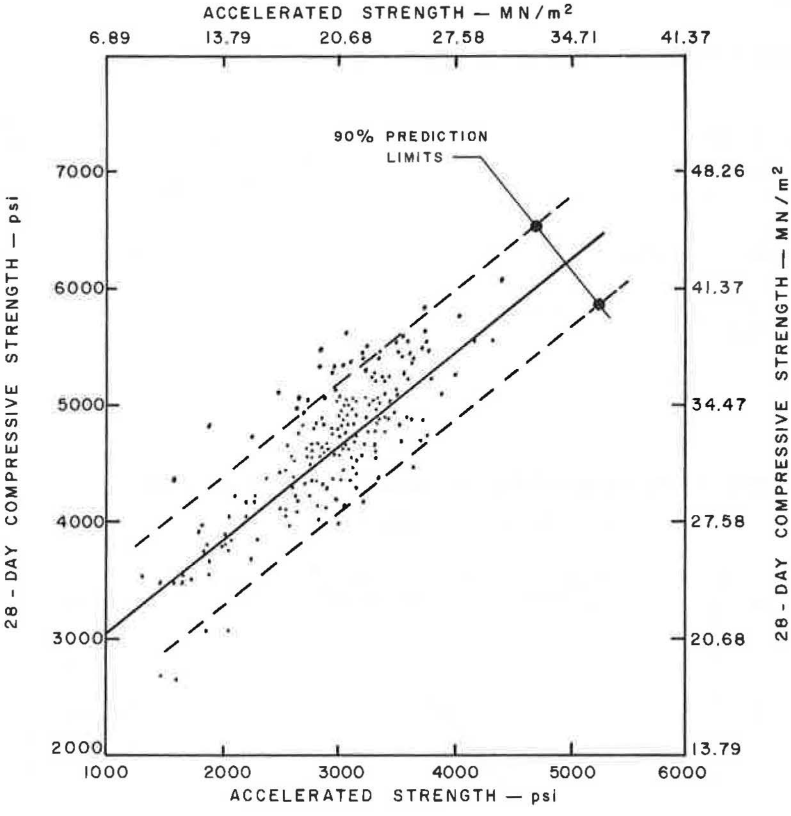


Figure 3. Seven-day versus 28-day strength results.

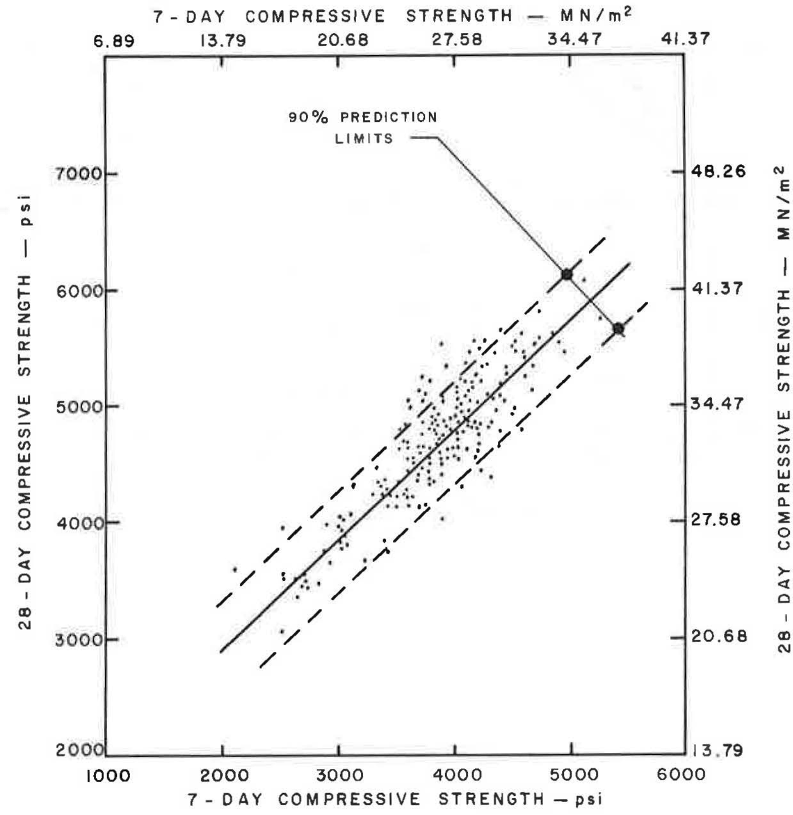


Table 8. Results of various field evaluations of accelerated strength.

Investigators	Method Used	Sample Size	Regression Equation	Correlation Coefficient	Standard Deviation (psi)
Smith and Tiede	Autogenous	—	$R_{28} = 1.35\ R_a + 1{,}180$ psi	—	301
Rodway and Ward	Modified boiling method	265	$R_{28} = 1.03\ R_a + 2{,}280$ psi	0.89	514
Malhotra and Zoldners	Modified boiling method	9	$R_{28} = 1.29\ R_a + 1{,}801$ psi	0.87	348
Bisaillon, Fréchette, and Keyser	Expanded polystyrene mold	213	$R_{28} = 0.81\ R_a + 2{,}263$ psi	0.80	333

Note: 1 psi = 6.9 kPa.

Figure 4. Accelerated and 28-day strength results from different studies.

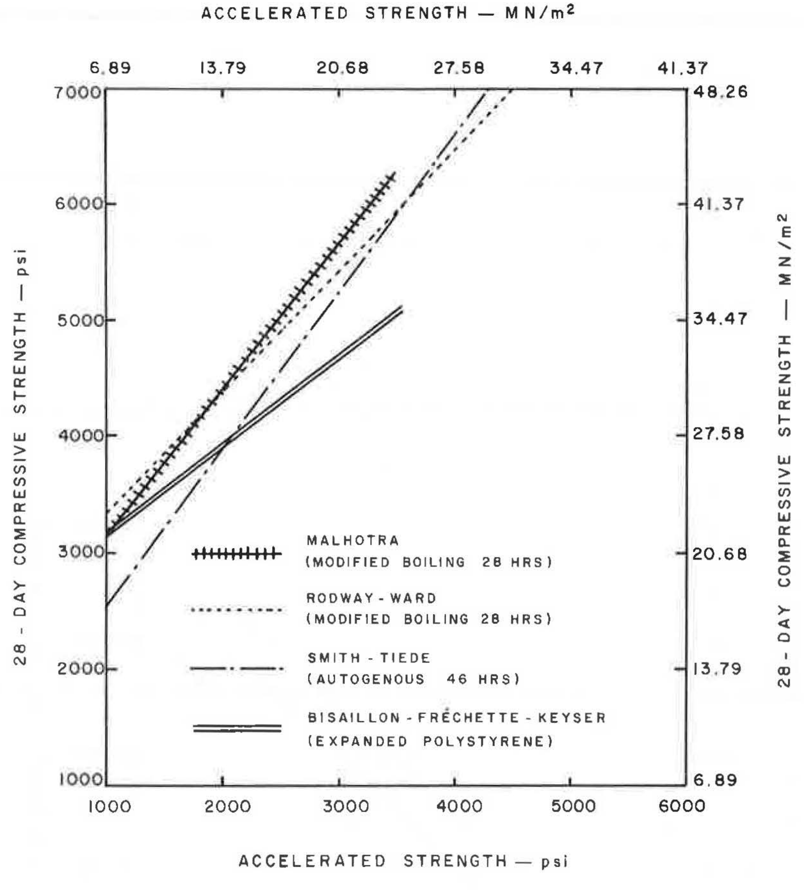
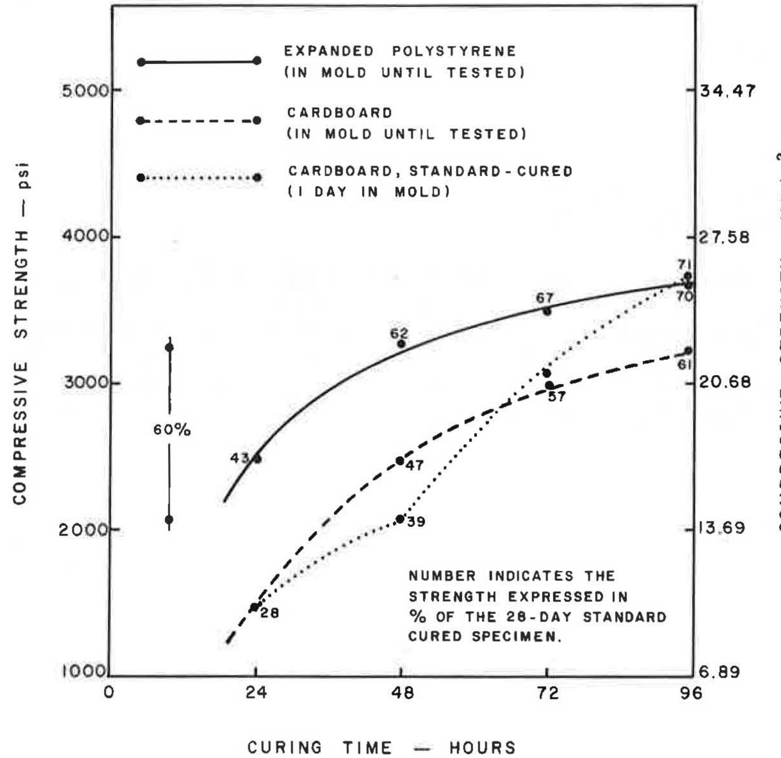


Table 9. Test variables investigated.

		Curing Time (hours)	
Series of Test	Mold Type	In Mold at Room Temperature	In Curing Room
1	Cardboard	24	24
2	Cardboard	Until tested	Nil
3	Expanded polystyrene	Until tested	Nil

Figure 5. Curing time versus strength for three curing conditions.



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3. V. M. Malhotra and N. G. Zoldners. Some Field Experience in the Use of an Accelerated Method of Estimate of 28-Day Strength. ACI Journal, Nov. 1969.
4. L. E. Rodway and M. A. Ward. Field Evaluation of an Accelerated Strength Test. Engineering Journal, Trans., Canadian Society for Civil Engineering, 1973.
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APPENDIX

STRENGTH GAIN OF CONCRETE CURED IN EXPANDED POLYSTYRENE VERSUS CARDBOARD MOLDS

Three series of concrete cylinders were molded and tested at 24, 48, 72, and 96 hours and at 28 days. The test variables investigated are given in Table 9.

The results of curing time and various compressive strengths are shown in Figure 5. Each point represents the average of six tests. The effect of self-cured accelerating action of the expanded polystyrene mold on the compressive strength of concrete at very early ages (less than 2 days) is clearly shown.

EARLY STRENGTH TEST FOR QUALITY CONTROL OF CONCRETE

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The time-honored approach to ensure quality of concrete is to take samples at the job site, to mold test cylinders, and then to cure the cylinders under standard conditions for 28 days before testing. The procedure is convenient, the test is simple, and the equipment is dependable. The disadvantage of the 28-day curing is the delay time that neither leads to the early correction of a material problem nor facilitates the prompt removal of defective concrete. Several researchers have demonstrated the feasibility of 24-hour tests to ensure quality control of concrete. Most early tests have been accelerated strength tests employing hot air, steam, or water for curing. The higher temperatures require special equipment, make the specimens difficult to handle, and introduce special problems when air-entraining agents and other additives are used. The merits of a simple 24-hour, 100 F (37.8 C) hot-water curing and testing procedure are described. Results achieved with the procedure are presented and analyzed. It is concluded that a hot-water curing temperature of 100 F (37.8 C) is adequate. The temperature is low enough to be completely safe for personnel involved and to permit careful handling of test specimens. The equipment required is simple and inexpensive.

•THE principal aspects of early strength testing of portland cement concrete are curing time, curing temperature, and experimental procedure. A curing time of 24 hours has considerable merit because it is convenient and leads to prompt results. The curing temperature is much more arbitrary. Initial tests were run to observe the effects of curing temperature on the level and reliability of 24-hour compressive strength.

TESTS PERFORMED

The study of curing temperatures was made by comparing results obtained from curing similar batches of concrete in different water temperatures. The strength correlations were achieved by curing and testing various grades of concrete by both the 28-day standard procedure and the accelerated hot-water procedure.

The concrete mix design used is given in Table 1. Tests were conducted by using hot-water curing temperatures of 90, 110, 130, and 150 F (32.2, 43.3, 54.4, and 65.6 C). Three 6-in.-diameter (15.2-cm) cylinders were molded from various batches of concrete and cured for 24 hours before testing. The 24-hour strength versus the curing temperature is shown in Figure 1. From the same batches of concrete, three additional cylinders were cast, cured for 28 days in a moist room, and then tested.

ANALYSIS OF RESULTS

The average compressive strengths of the cylinders cured at 90, 110, 130 and 150 F (32.2, 43.3, 54.4, and 65.6 C) were 1,879, 2,822, 2,990, and 3,107 psi (13, 19.5, 20.6, and 21.4 MPa) repectively. The results are shown in Figure 2.

Table 1. Concrete mix design.

Mix Designation	Water (lb/yd³)	Cement Factor (sacks/yd³)	Water-Cement Ratio (by weight)	Sand (lb/yd³)	Coarse Aggregate (lb/yd³)
A	325	7.30	0.488	1,320	1,710
B	325	7.27	0.477	1,200	1,820
C	325	6.08	0.570	1,310	1,800
D	325	5.20	0.667	1,400	1,770
E	325	4.53	0.766	1,490	1,740

Note: 1 lb/yd³ = 0.593 kg/m³.

Figure 1. Twenty-four-hour strength versus curing temperature.

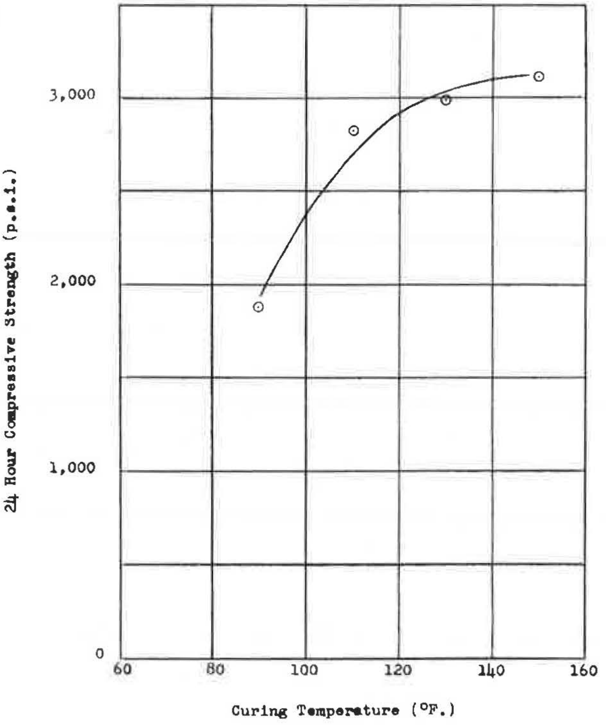
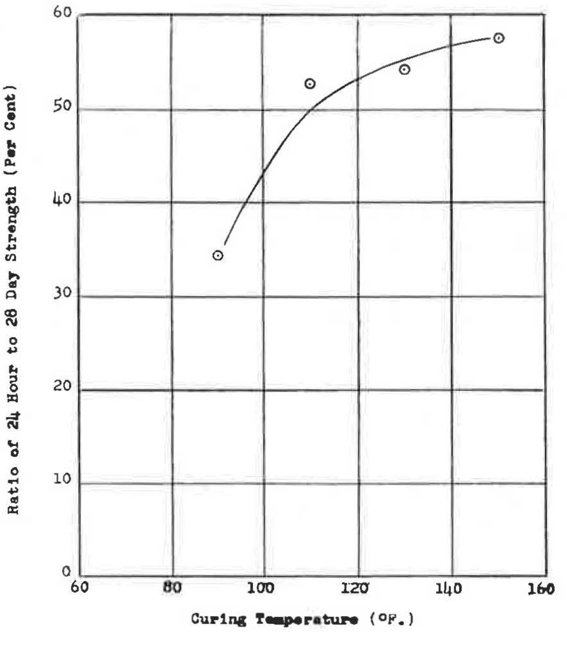


Figure 2. Percentage of strength gained in 24 hours versus curing temperature.



The tests demonstrate a substantial strength gain in 24 hours at a temperature as low as 100 F (37.8 C). Since these results were sufficiently interesting, we performed additional hot-water curing tests at 100 F (37.8 C). The 100 F (37.8 C) temperature is low enough to be regarded as a normal rather than an elevated temperature for accelerated curing. The merits of avoiding excessive temperatures are numerous and readily apparent. Since there is no threat to the safety of technicians, they are able to handle the specimens with care. When abnormal temperatures are not used, questions about thermal shock, the expansion of entrained air, and artificial changes in the hardening process do not arise. The equipment should add neither excessive heat nor humidity to the testing laboratory.

PROPOSED EARLY STRENGTH TEST

Procedure

The description of the procedure followed in conducting the early strength tests is recommended for further tests leading to the eventual adoption of standard specifications.

When mixing was completed, the properties of the plastic concrete were determined in accordance with ASTM specifications. Six standard 6 by 12-in. (15.2 by 30-cm) cylinders were prepared in cast-iron molds by following the procedure of ASTM C 192. The molds were capped with ½-in.-thick (1.27-cm) steel plates. They were turned on their sides and tapped several times at each end.

Fifteen min after the molding began, three cylinders were placed in the hot-water bath and three were placed in a moist room, where a standard temperature of 73.4 ± 3 F (23 C) was maintained. The cylinders placed in the hot-water bath were cured for 23½ hours. They were then removed, stripped, and tested at 24 hours according to ASTM C 39. The cylinders placed in the moist room were cured for 28 days and then removed, stripped, and tested in accordance with ASTM C 39.

The procedure produced cylinders with very good ends; no capping was needed.

Equipment

Perhaps the only equipment of interest is the hot-water bath, which is shown in Figure 3. The bath was simply constructed in our shop. The base to support the tank was constructed of 2 by 2 by ¼-in. (5.1 by 5.1 by 0.6-cm) steel angles. The tank was made of 0.13-in. (0.32-cm) steel plates welded together along their edges. The top was fitted with a wooden lid. The 16-ft^3 (0.45-m^3) tank is large enough to hold 14 cylinders. Circulation of the water is provided by a small electric impeller. Heat is supplied by a 6-in. (15.2-cm) ring gas burner. A solonoid valve is activated by a metallic expansion thermometer. The tank and its lid were insulated because the testing program included curing temperatures as high as 150 F (65.6 C). The tank was large enough for the tests conducted and provided temperature control for the water. Tanks of other sizes and shapes should perform equally well.

Additional equipment for producing and testing concrete conformed to ASTM C 31-66, C 39-66, C 143-66, C 192-62, and C 192-65. The cast-iron cylinder molds were fitted with ½-in. (1.3-cm) plates that could easily be swung into position and clamped. The end plates led to cylinders with good ends and eliminated the need for capping.

EARLY STRENGTH TEST EXPERIMENTS

Testing Program

Tests were performed to observe the merits of the 24-hour, 100 F (37.8 C) hot-water

Figure 3. Hot-water bath.

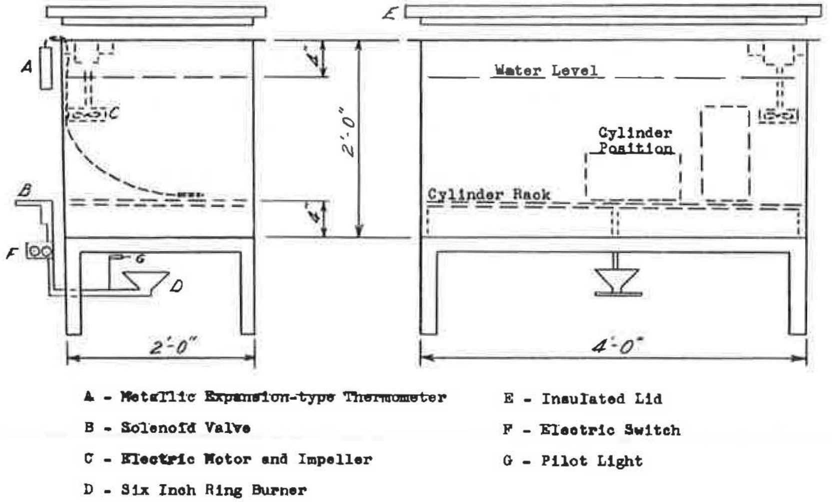
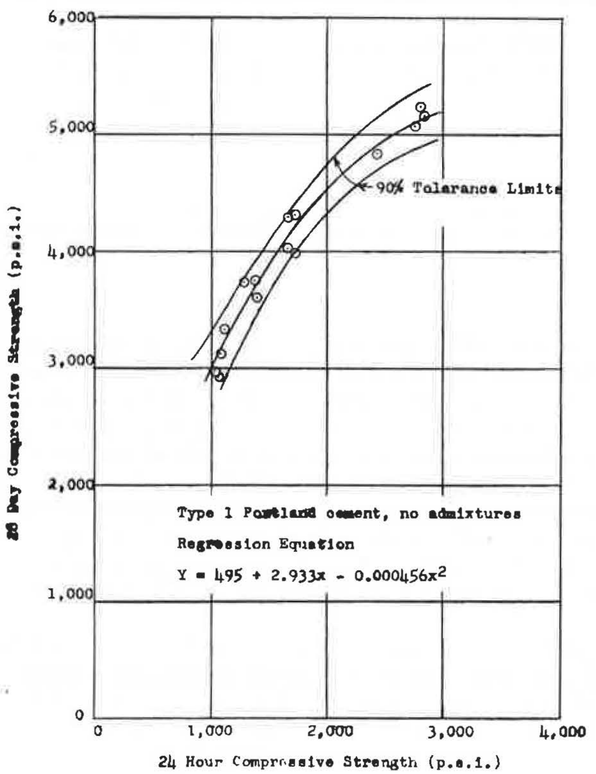


Figure 4. Twenty-four-hour strength versus 28-day strength.



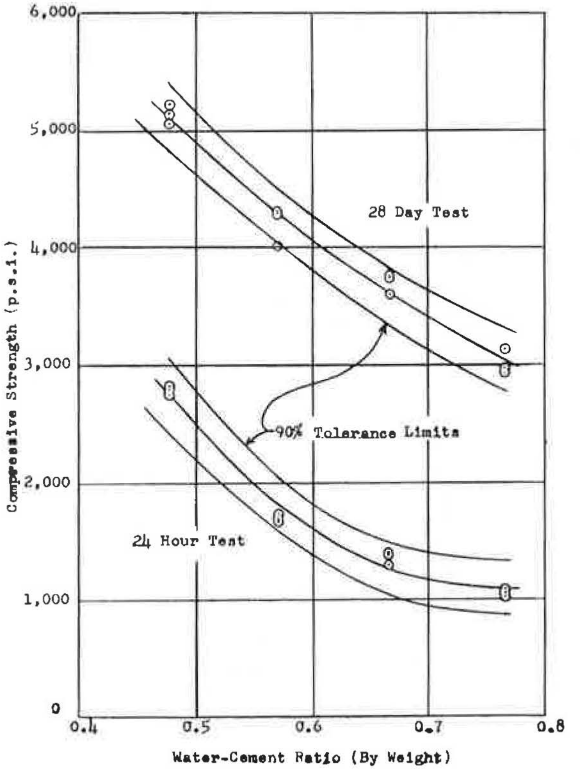


Figure 5. Compressive strength versus water-cement ratio.

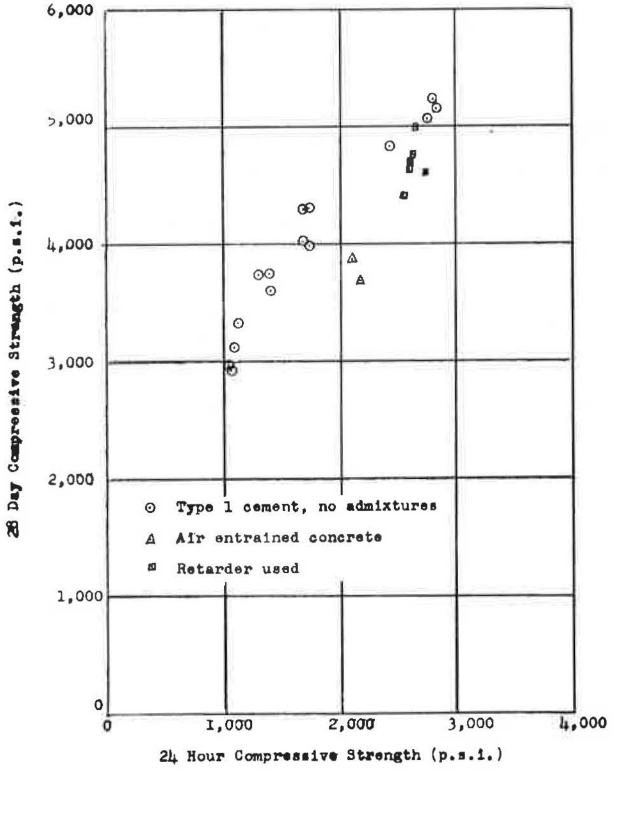


Figure 6. Effect of admixtures on strength correlation.

Figure 7. Effect of retarder on correlation between strength and water-cement ratio.

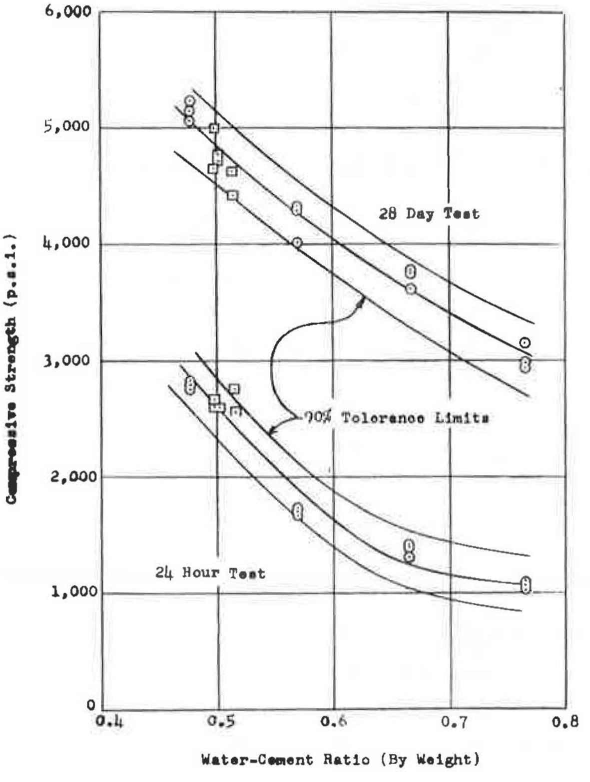


Table 2. Statistical analysis of results for each mix.

Mix Design	Test	Average Compressive Strength (psi)	Standard Deviation (psi)	Coefficient of Variation	Limits of Uncertainty[a] (psi)
B	24-hour	2,796.7	112.0	4.004	±73.9
B	28-day	5,147.6	170.4	3.310	±112.5
C	24-hour	1,691.4	78.8	4.658	±52.0
C	28-day	4,205.2	176.7	4.201	±116.6
D	24-hour	1,357.3	65.5	4.825	±43.2
D	28-day	3,701.0	121.1	3.272	±79.9
E	24-hour	1,058.0	31.9	3.015	±21.1
E	28-day	3,006.0	179.2	5.961	±118.3

Note: 1 psi = 6.9 kPa.

[a]P = 0.9.

curing. They were planned to determine (a) the correlation between the 24-hour test and the standard 28-day test (Figure 4) and (b) the correlation between 24-hour and 28-day compressive strength and the water-cement ratio (Figure 5).

Ninety standard cylinders were molded from the mixes without admixtures (Table 1). Thirty-six cylinders were prepared with a commercial retarder-densifier additive, and 12 were prepared with a commercial air-entraining agent. Figure 6 shows the effect of a retarder and an air-entraining agent on strength correlation, and Figure 7 shows the effect of a retarder on the correlation between strength and the water-cement ratio. The air content ranged between 6 and 7.5 percent. The procedure described previously was followed.

Analysis of Test Results

The compressive strength of each cylinder, the average strength, the standard deviations, and the coefficients of variation are given elsewhere (13). Table 2 gives the average compressive strengths, standard deviations, coefficients of variation, and limits of uncertainty obtained for each mix used. As the average 28-day strength increased from 3,006 to 5,148 psi (20.7 to 35.5 MPa), the strength gained in the 24 hours increased from 35 to 54 percent of the 28-day strength. It is noteworthy that the coefficients of variation (ratio of standard deviation to compressive strength) of the 24-hour strengths and 28-day strengths were much alike. The average values were 4.13 and 4.19 percent respectively.

CONCLUSIONS AND RECOMMENDATIONS

This experimental study leads to the following conclusions:

1. Twenty-four-hour compressive strengths can be used to predict 28-day strengths with good precision;
2. A hot-water curing temperature of 100 F (37.8 C) is adequate for a 24-hour early strength test; and
3. Test specimens of good quality can be produced with cast-iron molds equipped with steel end plates.

Based on the experiments described and additional research performed at the University of Delaware, we recommend the following:

1. A broadly based testing program leading to the development of an early strength test for concrete,
2. The use of hot-water curing,
3. A curing temperature low enough to ensure the careful handling of test specimens and the safety of the technicians involved, and
4. The eventual elimination of the 28-day test as a basis for design and quality control of concrete.

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STRENGTH TESTS AT EARLY AGES AND AT HIGH SETTING TEMPERATURES

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In some tropical countries, because of the favorable labor and weather conditions, concreting is frequently carried out 24 hours a day. This continual and high rate of placing of concrete requires a rapid control method for the estimation of potential strength. This paper presents a rapid control method by which compressive strength test results of concrete cubes 24 to 48 hours old will closely predict strengths at later ages. Test results of the cubes in which the setting temperatures are 113, 122, 131, 140, and 149 F (45, 50, 55, 60, and 65 C) are also presented. These setting temperatures considerably reduce the potential concrete strength. This reduction in strength is found to increase as the water-cement ratio and setting temperature increase.

•IN concreting operations, it is frequently advantageous or necessary to be able to predict the potential strength at any particular age or maturity almost immediately after the concrete has been placed. Normally, concrete cubes are tested at 7 and 28 days to determine whether the specified compressive strength at these two ages is met. However, where the placing of concrete has to be continuous and rapid and where large volumes are involved, this normal method of strength control is unsatisfactory because the test results are not available early enough to enable any unsatisfactory concrete to be removed easily or at low cost and to enable site adjustments of basic mix proportions to be effectively made if such changes are necessary.

RAPID CONTROL METHOD

To accommodate the needs precipitated by the continuous and rapid placing of concrete in large quantities, such as that which occurs in Malaysia, research programs on rapid control methods were undertaken. Ordman and Bondre (1) and King (2) carried out accelerated curing with oven heating and established an empirical relationship between the accelerated strength obtained by curing concrete cubes for 6 hours at a temperature slightly below 212 F (100 C) and the 28-day strength of the cubes normally cured in water. Akroyd (3) found that the ratio of the strength of cubes that were cured at 140 F (60 C) for 7 hours to the strength at 7 days of normal curing is the same as the ratio of the 7-day strength to that at 28 days of normal curing. King (4) reported that testing cubes after curing in water at 131 F (55 C) would enable the prediction of the 28-day strength with reasonable accuracy. He produced graphs relating these two strengths.

These investigators cured cubes at some fixed temperature and duration and related the strength of a single maturity to the 7 or 28-day strength.

Ever since it was realized that the strength of concrete increases as time or maturity increases, attempts have been made to express this relation in the form of an equation so that strength may be predicted without the need to perform long-term tests. It has been shown (5) that the relation between strength and age or maturity is hyperbolic, that is,

$$D/q = mD + C \qquad (1)$$

where q is the compressive strength at age D or maturity M, and C and m are constants. If D/q is plotted against an abscissa of D, equation 1 becomes a straight line, the inverse slope (1/m) of which gives the maximum strength the concrete will attain with age.

Equation 1 will therefore enable the maximum strength that a concrete will finally attain and the strength at any specified age or maturity to be predicted from the test results obtained at an early age. This equation was, however, established with test results of specimens that were predominantly more than 3 days old. Therefore, it seems necessary to test the validity of equation 1 for the prediction of concrete strength with test results obtained at much earlier ages than 3 days and to determine what accuracy can be attained when the equation is used for predicting concrete strength by using the results at early maturities. It would, therefore, be necessary to start testing over a wide spectrum of maturities at as early a maturity as possible and to extend the tests to higher maturities. The lowest maturity will seem to be that at which the cubes have attained sufficient strength to withstand handling without physical damage so that reliable results can be produced. Under tropical conditions, this appears to be the age at which test cubes are normally demolded, that is, after an age of about 24 hours in an average ambient temperature of 86 F (30 C).

Cubes made from the same batch were tested at ages from 23 to 1,009 hours (about 42 days). Table 1 gives the results of set J, for which the mix was 1:2:4 by weight; the water-cement ratio was 0.45 (ordinary portland cement, river sand, and limestone were used); and the water curing temperature was 82.4 ± 33.8 F (28 ± 1 C). About forty-two 4-in. (10-cm) cubes were made from each batch, and the crushing strength given in Table 2 represents the mean strength of two test cubes.

In determining the linearity among the plots of D/q against an abscissa of D, the method of least squares is used. The closeness to 1 of the value of the product moment correlation coefficient confirms the high degree of linear association among the variates.

On the basis of the strengths at 23 and 29 hours, the predicted maximum strength of the concrete is 6,803 psi (46.9 MPa); on the basis of those at 23, 29, 36, and 47 hours, it is 7,194 psi (49.6 MPa). The mean value for the predicted maximum strength is 7,474 psi (51.5 MPa). The strength results at ages of within 48 hours, therefore, give a prediction that is reasonably close to that based on all 21 test results at ages from 23 to 1,009 hours.

For rapid-hardening cement, the test results at ages of up to 48 hours also give a close prediction of the maximum strength that will ultimately be attained. Table 2 gives the results of set P based on a rapid-hardening portland cement. The mix was 1:2:4 by weight; the water-cement ratio was 0.55 (rapid-hardening cement, river sand, and limestone were used); and the water curing temperature was 82.4 ± 33.8 F (28 ± 1 C).

On the basis of the test results at ages of 23, 29, 35, and 47 hours (Table 2), the following equation relating age D to crushing strength q is obtained:

$$D/q = 0.000153D + 0.000935 \qquad (2)$$

Using these four test results, that is, using equation 2 to predict the strength at, say, 695 hours, we have

$$695/q = 0.000153(695) + 0.000935 \qquad (3)$$

As a result, 6,479 psi (44.7 MPa) is the strength of the concrete when its age reaches 695 hours. This is a close prediction of the actual average crushing strength of 6,132 psi (42.3 MPa) when the cubes were tested at an age of 695 hours.

Equation 2 predicts a crushing strength of 6,497 psi (42.3 MPa) at 1,031 hours (about 42 days). The actual average crushing strength at this age was 6,584 psi (45.3 MPa).

This series of tests covered water-cement ratios of 0.45, 0.50, and 0.55. All the

Table 1. Compression test results for concrete test cubes made of ordinary portland cement.

Age (hours)	Average Crushing Strength (psi)	Correlation Coefficient	Constant	1/m (psi)
23	3,014			
29	3,409	1.0000	0.000147	6,803
36	3,658	0.9974	0.000171	5,848
47	4,309	0.9897	0.000139	7,194
53	4,340	0.9937	0.000147	6,803
71	5,132	0.9919	0.000131	7,634
78	5,035	0.9944	0.000136	7,353
95	5,712	0.9934	0.000127	7,874
106	5,663	0.9956	0.000129	7,752
132	5,589	0.9944	0.000138	7,246
144	5,519	0.9961	0.000138	7,246
156	6,321	0.9955	0.000134	7,463
167	6,349	0.9961	0.000132	7,576
193	6,633	0.9963	0.000129	7,752
239	6,643	0.9975	0.000129	7,752
336	6,765	0.9985	0.000132	7,576
432	6,927	0.9991	0.000133	7,519
504.6	7,203	0.9993	0.000131	7,634
672	7,870	0.9979	0.000125	8,000
840	7,980	0.9982	0.000121	8,265
1,009	7,777	0.9989	0.000122	8,197

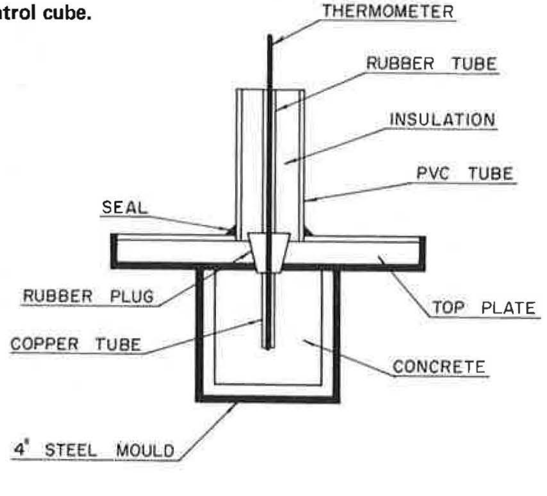
Note: Mean value of inverse of slope = 7,474 psi (51.5 MPa). 1 psi = 6.9 kPa.

Table 2. Compression test results for concrete test cubes made of rapid-hardening portland cement.

Age (hours)	Average Crushing Strength (psi)	Correlation Coefficient	Constant	1/m (psi)
23	5,240			
29	5,443	1.0000	0.000156	6,410
35	5,445	0.9990	0.000170	5,882
47	5,849	0.9978	0.000153	6,536
53	5,856	0.9989	0.000153	6,536
73	5,979	0.9996	0.000155	6,452
98	6,664	0.9961	0.000140	7,143
124	6,671	0.9980	0.000139	7,194
143	6,135	0.9958	0.000148	6,757
167	6,808	0.9962	0.000144	6,944
191	6,520	0.9973	0.000145	6,897
215	6,356	0.9976	0.000149	6,711
335	5,824	0.9952	0.000163	6,135
455	6,335	0.9973	0.000160	6,250
527	6,139	0.9983	0.000161	6,211
695	6,132	0.9991	0.000161	6,211
863	6,027	0.99932	0.000164	6,098
1,031	6,584	0.99845	0.000159	6,289

Note: 1 psi = 6.9 kPa.

Figure 1. Control cube.



test results produced similar levels of accuracy. From these tests, it could be concluded that test results obtained from cubes at ages of between 24 and 48 hours can produce close predictions of strength at higher ages. This, therefore, provides a rapid and reliable method of strength control.

HIGH SETTING TEMPERATURE

In tropical countries, like Malaysia, where aggregates, formwork, and reinforcing bars are frequently exposed to the sun, mixing, placing, and setting temperatures can be high. Often the supply situation of cement has been such that cement fresh from the cement works has to be used before it has the time to cool. Placing temperatures as high as 131 F (55 C) have occurred. It has been known that, although high curing temperatures would be beneficial because early strength is attained without subsequent adverse effects, placing and setting temperatures higher than 95 F (35 C) would reduce the maximum strength that the concrete would otherwise attain. [The reduction in strength when concrete is allowed to set at a temperature of 131 F (55 C) is also examined.] This reduction in strength is measured by comparing the test results of the cubes for which the setting temperature is 131 F (55 C) with those cubes from the same batch that are allowed to set at an average ambient temperature of 86 F (30 C) and that are then normally cured in water at 82.4 F (28 C).

Thirty-three 4-in. (10-cm) cubes were made from each batch; 11 for setting and curing at a temperature of 131 F (55 C), and the remaining 22 cubes were for normal curing in water at 82.4 ± 33.8 F (28 ± 1 C) after they had been set in the molds at an average ambient temperature of 86 F (30 C). The concrete was placed in the molds in two layers; each layer was vibrated for 1 min.

Setting Temperature of 131 F (55 C)

The 11 cubes of each batch were immediately leveled, and each mold was then covered with a watertight top plate. The molds were then placed in a tank into which water previously heated to a temperature of 131 F (55 C) was poured to completely submerge the molds. The warm water was maintained at 131 F (55 C), and the thermometers that were inserted into the center of two control cubes (Figure 1) were read periodically throughout the entire period of setting and curing. These cubes were tested at 3, 6, 9, 12, 15, 18, and 27 hours. Age was reckoned from the moment the warm water was introduced into the tank.

Normal Curing

The remaining 22 cubes from the same batch were covered with moist sacks for 24 hours in an average ambient temperature of 86 F (30 C). They were all demolded at an age of 24 hours and then cured in water maintained at a temperature of 82.4 ± 33.8 F (28 ± 1 C) until they were required for testing.

These cubes were tested at 1, 3, 7, 14, 28, 35, and 42 days. The test results of various sets are given in Table 3. All the mixes were 1:2:4 by weight. Batu Caves ¾-in. (19-mm) limestone, river sand, and a locally manufactured portland cement complying with the requirements of British Standard 12 were used.

Figure 2 shows D/q versus D for a set of cubes in which the setting temperature was 131 F (55 C). The values of the product moment correlation coefficient obtained from the test results of all the sets that were set in warm water are very close to 1. There is, therefore, a high degree of linear association between D/q and D. $1/m_a$ of the setting in warm water (Table 4) gives the inverse slopes, that is, the values of $1/m_a$ computed on the basis of the strength-age results obtained from the test cubes for ages from 3 to 27 hours. It is found that the values of $1/m_a$ decrease as the water-cement ratio increases.

Table 3. Results of cubes tested at 1, 3, 7, 14, 28, 35, and 42 days.

Water-Cement Ratio	Set No.	Warm-Water Temperature (C)	Water Temperature in Normal Curing (C)
0.40	3	55	28
0.45	2	55	28
0.50	1	45	28
0.50	1	50	28
0.50	5	55	28
0.50	1	60	28
0.50	1	65	28
0.60	3	55	28

Note: 1 C = (F - 32)/1.8

Figure 2. D/q and q versus D.

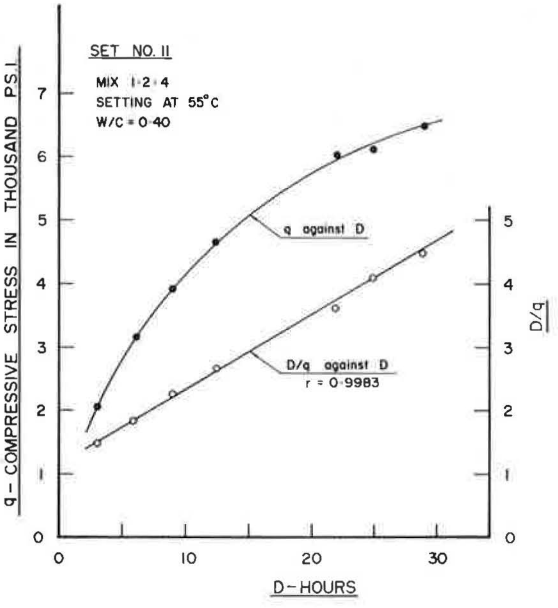


Table 4. Comparative values of $1/m_a$ and $1/m_n$.

Set No.	Water-Cement Ratio	Setting in Warm Water: Temperature (C)	Setting in Warm Water: $1/m_a$ (psi)	Setting in Ambient Temperature, 30 C, $1/m_n$ (psi)	Ratio of $1/m_a$ to $1/m_n$	Mean Ratio
4	0.40	55	9,259	9,709	0.95	
5	0.40	55	9,091	8,929	1.02	0.95
11	0.40	55	8,760	10,000	0.88	
16	0.45	55	7,107	9,015	0.79	0.80
17	0.45	55	8,053	9,902	0.81	
1	0.50	55	5,848	8,264	0.71	
2	0.50	55	4,785	6,494	0.74	
3	0.50	55	6,061	7,874	0.77	0.79
7	0.50	55	6,410	7,645	0.84	
8	0.50	55	5,714	6,525	0.88	
6	0.60	55	4,255	5,400	0.79	
14	0.60	55	4,320	6,121	0.71	0.73
15	0.60	55	4,135	5,724	0.72	
12	0.50	45	5,720	7,189	0.80	0.80
9	0.50	60	5,988	7,600	0.79	0.79
13	0.50	65	5,910	8,072	0.73	0.73

Note: 1 psi = 6.9 kPa. 1 C = (F -32)/1.8

The relation between D/q and D for the cubes that were normally cured in water at a temperature of 82.4 ± 33.8 F (28 ± 1 C) after setting at an average ambient temperature of 86 F (30 C) is also linear for all the water-cement ratios concerned.

Figure 3 shows the maximum stress computed from the inverse slope versus the water-cement ratio for the cubes that had set at ambient temperature and for those that had set in warm water. This relation between strength and cement-water ratio is approximately linear.

It is clear from Table 4 that, for every set of the test results, the value of 1/m for the cubes in which the setting was in warm water is lower than that obtained with the cubes of the same batch that had set at an average ambient temperature of 86 F (30 C). Let R be the ratio of $1/m_s$, the inverse slope of the plot of D/q against an abscissa of D obtained from the cubes in which the setting was in warm water, to $1/m_n$, the inverse slope obtained from the test results of those cubes that were normally cured. The values of R are less than 1 (Table 4), and this indicates that the higher setting temperature has considerable adverse effect on concrete strength. For example, a setting temperature of 131 F (55 C) reduces the maximum strength that the concrete will ultimately attain by 27 percent for the 1:2:4 mix with a water-cement ratio of 0.60. This is a significant reduction.

The test results show that the value of R decreases as water-cement ratio increases. The values of R are 0.95, 0.80, 0.79, and 0.73 for water-cement ratios of 0.40, 0.45, 0.50, and 0.60 respectively.

For a water-cement ratio of 0.50, the values of R are 0.80, 0.80, 0.79, and 0.73 for setting temperatures of 113, 131, 140, and 149 F (45, 55, 60, and 65 C) respectively. The maximum strength that the concrete will attain, therefore, decreases as setting temperature increases.

Development of Concrete Strength

The steepness of the strength-maturity curves at the beginning indicates very rapid initial growth of strength. This rate of growth in strength decreases as age or maturity increases until a finite value is ultimately reached. In equation 1, when q is differentiated with respect to D,

$$dq/dD = C/(mD + C)^2 \tag{4}$$

That is, the rate at which strength changes with age or maturity is equal to $C/(mD + C)^2$. When D is large, $dq/dD \rightarrow 0$; that is, after an appreciable time or maturity, the gain in strength with age becomes insignificant. The test results show that as much as 88 percent of the final concrete strength is attained at an age of 28 days for those cubes normally cured in water. This earlier attainment of final strength is typical of concrete placed under tropical conditions. The increases in strength with respect to age are, therefore, considerably less than those permitted in British Standard CP 114(1957) or in CP 110(1971).

When D is zero, that is, at the moment when the concrete begins to develop its strength,

$$dq/dD = 1/C \tag{5}$$

The value of 1/C is, therefore, the initial and also the maximum rate at which concrete develops its strength. This rate decreases as age or maturity increases. Table 5 gives the values of 1/C for the tests carried out. The value of 1/C for the cubes that had set at 131 F (55 C) is much higher than that for the cubes of the same batch that were allowed to set at an average ambient temperature of 86 F (30 C). The value of

Figure 3. Average predicted maximum compressive stress versus water-cement ratio.

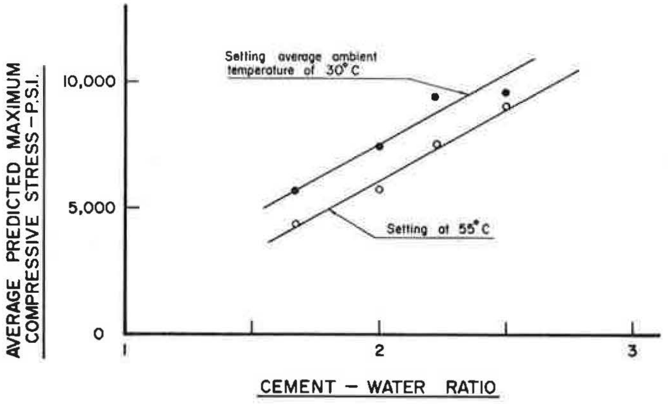


Table 5. Maximum rate of development of concrete strength.

Set No.	Water-Cement Ratio	Setting at 30 C: C × 10^{-4}	Setting at 30 C: Average C × 10^{-4}	Setting at 30 C: 1/C	Setting at 55 C: C × 10^{-4}	Setting at 55 C: Average C × 10^{-4}	Setting at 55 C: 1/C
4	0.40	6.43	5.01	199.6	1.07	1.14	877.2
5	0.40	3.91			1.17		
11	0.40	4.69					
16	0.45	5.70	5.69	175.7	0.84	1.22	819.6
17	0.45	5.67			1.59		
1	0.50	6.96	7.74	129.2	2.05	2.19	456.6
2	0.50	8.78			2.40		
3	0.50	7.92			1.98		
7	0.50	7.10			2.15		
8	0.50	7.94			2.35		
6	0.60	13.87	13.87	72.1	3.02	3.17	315.5
14	0.60	12.97			3.18		
15	0.60	14.78			3.31		

Note: 1 C = (F - 32)/1.8

1/C increases as the water-cement ratio for both the setting temperatures increases. The test results show that the higher the value of 1/C is, that is, the higher the initial rate of growth of strength is, the lower the maximum strength finally attained by the concrete will be.

CONCLUSIONS

This paper reports on a limited pilot study that is being extended to cover a larger range of water-cement ratios and setting temperatures and types of cement and aggregates. In particular, it will be expanded to determine whether there is in fact a critical setting temperature below which concrete strength will not be adversely affected. It seems necessary to extend the tests of the cubes that set at the high temperatures to higher maturities or ages than those reported in this paper to ascertain whether there is any possible recovery of concrete strength with time.

On the basis of the test results, the following conclusions have emerged:

1. The compressive test results obtained with cubes at ages between 24 and 48 hours can produce a close prediction of compressive strength at later ages.
2. Setting temperatures of 113 to 140 F (45 to 60 C) have significant adverse effects on concrete strength. The reduction in strength increases as the water-cement ratio increases.
3. The higher the initial rate of growth of concrete strength is, the lower the maximum strength finally attained by the concrete will be.
4. Under the higher ambient and water temperatures in the tropics, there is an earlier attainment of final concrete strength.

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ACCELERATED TESTING FOR PREDICTION OF 28-DAY STRENGTH OF CONCRETE

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There is a need for timely information on strength of concrete in construction works to help achieve better quality control. In India, not much work on accelerated strength testing of concrete has been reported, and there is a need for a systematic study of such testing by using locally available cements. This paper gives an account of an experimental study of accelerated testing of concrete for prediction of strength by using two methods based on suitable modifications of the British accelerated testing committee method and the Canadian modified boiling method. Concrete mixes with strengths ranging from 150 to 600 kgf/cm^2 (15 to 60 MPa) were tested. Test results, which were correlated by regression analyses, show that accelerated testing (hot-water curing for 3½ hours at 85 C) can be used to accurately predict 28-day concrete strengths.

•CONVENTIONAL practice for specification of concrete consists of denoting the 28-day compressive strength of 15 by 30-cm cylinders or 15-cm cubes cast, moist cured, and tested under controlled conditions. However this duration is too long in the context of current trends in new design concepts, increased pace of construction, and timely control of concrete quality. The need for an early determination of the concrete strength with satisfactory reliability has long been felt by many as evidenced by the research work published by many institutions all over the world during the past few decades. Different accelerated tests are being used in different countries for the prediction of the 28-day compressive strength, and these are available either as departmental or committee recommendations or incorporated as specification in the codes (1, 2, 3, 4, 5, 6, 7, 8). In general, these methods use the application of external heat or conservation of the heat of hydration for the accelerated strength development, and the methods differ mainly in procedural details.

In India, accelerated strength testing of concrete has not been widely attempted, and there is a need for a systematic study of such testing using locally available cements. Rehsi, Garg, and Kalra (9) have studied the suitability of the test method, recommended by the British accelerated testing committee (BATC), for application of 28-day compressive strengths of 100 kgf/cm^2 (10 MPa) to 250 kgf/cm^2 (25 MPa) to concretes of grades M 100 to M 250 (11 cement samples from the various factories of the Associated Cement Companies Ltd., Bombay). Two mixes with water-cement ratios of 0.4 and 0.6 and aggregate-cement ratios of 3.03 and 6.09 were used for the tests. It was concluded that the BATC method could be adopted for predicting the 28-day strength of the concrete mixes tried.

In this paper, a detailed laboratory study is reported on accelerated strength testing of concrete with strengths ranging from 150 to 600 kgf/cm^2 (15 to 60 MPa) by two different methods. The results of the tests have been assessed with the aid of suitable regression analyses.

EXPERIMENTAL PROGRAM

Materials and Equipment

Ordinary portland cements from three manufacturers (A, B, and C) in South India were used. The physical properties of these cements, obtained from standard tests [Indian Standard 4031 (1968)], are given in Table 1.

Crushed granite aggregate [maximum size, 8 in. (20 mm)] and river sand were used. Their gradations are given in Table 2.

Normal potable water was used for mixing and curing.

Various mixes were adopted with water-cement ratios ranging from 0.30 to 0.68; the values of aggregate cement ratios were 3.0, 4.5, 6.0, and 9.0. For the most part, the combined aggregate consisted of two parts of coarse aggregate and one part of fine aggregate by weight, and this resulted in a good continuous grading for the mix. The 28-day strengths of the mixes ranged from about 150 kgf/cm^2 to 600 kgf/cm^2 (15 to 60 MPa).

A thermostatically controlled, 87 by 60 by 38-cm-high water bath was used for accelerated curing. The bath was equipped with a main heater of adequate capacity and a secondary heater of 200 W to be operated by thermostatic control. The water level in the bath was maintained to provide for a volume of 0.05 m^3 of water for each specimen (15-cm cube). To maintain a uniform temperature, the water was continuously stirred.

Methods of Testing

Two methods of accelerated testing based on the BATC method (1) and the Canadian modified boiling method (6) were selected for the tests, and slight modifications in procedural details were made to suit the available facilities.

The modified methods adopted are named method 1 and method 2 respectively. The details of the basic methods and the modifications adopted for the tests are given in Table 3.

Test Procedure

Standard 15-cm cube specimens were cast in steel molds on a vibrating table. Twelve specimens were prepared from each concrete mix, of which four specimens were subjected to accelerated testing in the water bath and four each were normally cured in water for the 7-day and 28-day strength tests. For accelerated testing by method 1, the steel molds containing the concrete specimens were covered by 7-mm-thick steel plates, and these were cured in the water bath at the specified temperature. For method 2, the molds were covered with damp sacks for 24 hours after which the specimens were demolded and cured in the water bath at 85 C. Method 1 was attempted with 1 cement (cement A), and 25 sets of 15-cm cubes were cast, 12 cubes per set of concrete mix. Method 2 used two cements (cements B and C), and 16 sets of mixes were cast with each cement. The details of the mixes and the strengths obtained (average of four cubes each) are given in Tables 4, 5, and 6.

In Tables 4, 5, and 6, R_1 is the value of the 1-day accelerated strength test by the particular test method; R_7 and R_{28} are the 7 and 28-day strengths of normally cured specimens respectively.

Table 1. Physical properties of cements.

Cement	Setting Time (min)		Fineness[a] (cm^2/g)	Compressive Strength (kgf/cm^2)	
	Initial	Final		3 Days	7 Days
A	40	115	2740	207	255
B	86	123	3240	212	300
C	70	135	3010	164	227

Note: 1 kgf/cm^2 = 0.098 MPa.
[a]Blaine's.

Table 2. Sieve analysis of aggregates.

Sieve Size	Percent Passing: Coarse Aggregate	Percent Passing: Sand	Sieve Size	Percent Passing: Coarse Aggregate	Percent Passing: Sand
20 mm	100	—	120 μm		63.1
10 mm	31.5	—	60 μm		43.8
480 μm	0	100	30 μm		11.1
240 μm		81.5	15 μm		0

Table 3. Details of accelerated test methods.

Item	BATC Method (hours)	Method 1 (hours, min)	Canadian Boiling Method (hours)	Method 2 (hours, min)
Commencement of curing[a]	½	30, ±5	24	24, ±15
Duration of curing	24[b]	23, ±15[c]	3½[d]	3½[e]
Commencement of testing[f]	½	½, ±10	1	1, ±10
Total duration for test	25	24	28½	28½

[a]Time after curing.
[b]At 55 C.
[c]At 60 ± 1C.
[d]At 100 C.
[e]At 85 ± 2 C.
[f]Time after completion of curing.

Table 4. Compressive strength results based on method 1 and cement A.

Set No.	Mix Proportions[a] (by weight)	Water-Cement Ratio (by weight)	Compressive Strength (kgf/cm^2): R_1	R_7	R_{28}
1	1:3:6	0.55	115.0	159.0	248.0
2	1:3:6	0.60	108.5	141.0	208.0
3	1:3:6	0.65	98.5	117.0	179.0
4	1:3:6	0.68	79.0	109.0	160.0
5	1:2:4	0.45	212.0	312.0	439.0
6	1:2:4	0.50	194.0	255.0	398.0
7	1:2:4	0.55	162.0	218.0	351.0
8	1:2:4	0.60	131.0	186.5	306.5
9	1:2:4	0.64	123.0	192.0	296.5
10	1:1.5:3	0.36	290.0	378.0	588.0
11	1:1.5:3	0.40	261.0	370.0	525.0
12	1:1.5:3	0.45	185.5	277.0	417.0
13	1:1.5:3	0.50	180.0	270.0	411.0
14	1:1.5:3	0.55	139.0	232.0	334.5
15	1:1.5:3	0.66	97.0	170.0	235.0
16	1:1:2	0.30	330.0	431.0	573.0
17	1:1:2	0.33	290.0	383.0	565.0
18	1:1:2	0.36	281.0	355.0	484.5
19	1:1:2	0.40	208.0	301.0	454.5
20	1:1:2	0.45	153.0	245.5	371.5
21	1:0.81:1.84	0.30	354.0	433.0	634.5
22	1:1.16:2.32	0.32	346.5	414.0	573.0
23	1:1.16:2.32	0.35	307.0	391.0	511.0
24	1:1.12:2.44	0.33	339.0	411.0	560.5
25	1:1.12:2.44	0.35	315.5	373.0	512.0

Note: 1 kgf/cm^2 = 0.098 MPa.
[a]Cement:sand:aggregate.

Table 5. Compressive strength results based on method 2 and cement B.

Set No.	Mix Proportions[a] (by weight)	Water-Cement Ratio (by weight)	Compressive Strength (kgf/cm^2)		
			R_1	R_7	R_{28}
1	1:3:6	0.60	133.0	202.5	310.0
2	1:3:6	0.65	116.0	183.5	290.5
3	1:3:6	0.68	101.0	161.5	266.0
4	1:2:4	0.50	185.5	283.0	400.0
5	1:2:4	0.55	172.6	281.5	373.0
6	1:2:4	0.60	145.5	243.5	330.5
7	1:2:4	0.64	118.5	198.0	285.5
8	1:1.5:3	0.36	305.0	478.0	582.5
9	1:1.5:3	0.40	282.0	446.0	524.5
10	1:1.5:3	0.45	190.0	307.5	405.5
11	1:1.5:3	0.50	190.0	296.5	399.5
12	1:1.5:3	0.55	168.5	276.0	341.0
13	1:1:2	0.30	347.5	476.0	568.0
14	1:1:2	0.33	286.0	450.5	560.5
15	1:1:2	0.36	256.0	410.0	523.0
16	1:1:2	0.40	255.0	412.5	481.0

Note: 1 kgf/cm^2 = 0.098 MPa.

[a]Cement:sand:aggregate.

Table 6. Compressive strength results based on method 2 and cement C.

Set No.	Mix Proportions[a] (by weight)	Water-Cement Ratio (by weight)	Compressive Strength (kgf/cm^2)		
			R_1	R_7	R_{28}
1	1:3:6	0.60	113.5	195.5	275.0
2	1:3:6	0.65	101.0	172.5	259.5
3	1:3:6	0.68	93.0	147.5	220.0
4	1:2:4	0.50	154.0	253.5	345.0
5	1:2:4	0.55	148.5	241.5	323.0
6	1:2:4	0.60	115.5	190.0	280.5
7	1:2:4	0.64	96.0	155.5	236.5
8	1:1.5:3	0.36	220.5	330.0	464.5
9	1:1.5:3	0.40	210.5	327.0	435.5
10	1:1.5:3	0.45	151.0	246.0	330.0
11	1:1.5:3	0.50	139.0	229.5	318.0
12	1:1.5:3	0.55	110.0	195.0	270.5
13	1:1:2	0.30	261.5	411.0	507.5
14	1:1:2	0.33	260.5	405.5	484.5
15	1:1:2	0.36	223.0	337.0	456.0
16	1:1:2	0.40	187.0	287.0	405.0

Note: 1 kgf/cm^2 = 0.098 MPa.

[a]Cement:sand:aggregate.

ANALYSIS OF RESULTS

Accelerated Test Method 1

Curves of the correlation between R_1 and R_7 and between R_1 and R_{28} are shown in Figures 1 and 2. These are parabolic in nature, and the best fit for the parabolic curve was obtained by means of statistical methods (10). The equations obtained for the best fitting curves are also shown in Figures 1 and 2. The standard errors or prediction for R_7 and R_{28} were 14.2 kgf/cm^2 (1.4 MPa) and 27.5 kgf/cm^2 (2.7 MPa) respectively.

Accelerated Test Method 2

Based on Figures 1 and 2, the correlations were almost linear, and the plots for both cements B and C nearly coincide. Hence a single linear correlation, based on the results for cements B and C (total population), was then attempted. By statistical methods (11), the regression line that gave the best linear fit was determined. The results of this analysis are shown in Figures 3 and 4. The standard error of prediction is small, 8.33 kgf/cm^2 (0.8 MPa) and 10.34 kgf/cm^2 (1.0 MPa), for prediction of R_7 and R_{28} values from R_1 values. The 95 percent confidence limits for the correlation are also shown in Figures 3 and 4.

DISCUSSION OF RESULTS

Accelerated Test Method 1

Almost all the results shown in Figures 1 and 2 are within the ±10 percent error limits. The scatter of results is relatively more with the high-strength concretes, i.e., for 28-day strengths of 500 kgf/cm^2 (50 MPa) and more. But the standard error of prediction is small. The parabolic correlation is compared with that obtained by the BATC method (Figure 5). This shows a similar trend between the two sets of results up to concrete strengths of about 500 kgf/cm^2 (50 MPa). The method appears to be quite useful for prediction of strength but involves a procedure of maintaining a constant temperature bath at 60 C for 23 hours and requires the use of the mold and a cover plate for the curing.

Accelerated Test Method 2

This method gives a linear correlation for prediction of 7-day or 28-day strengths (Figures 3 and 4), and the standard error of prediction is considerably lower than that of method 1. The 95 percent confidence limits are close together (Figures 3 and 4). The linear character of the relationship and the coefficient of correlation obtained by this method compare well with the results obtained by the Canadian modified boiling method (Figure 6) (6). The correlation obtained appears to be independent of the source of the cement, cement content, and water-cement ratio. The method has good potential for application and can give accurate results. A more representative character of the linear relationship can be obtained by undertaking a large number of laboratory and field tests, say, about 500 sets of results, with portland cements from a number of different sources in the country.

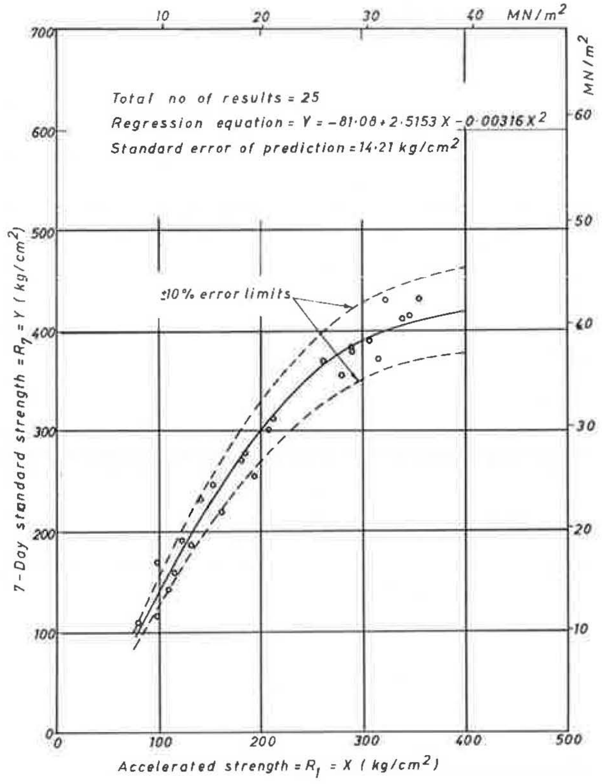


Figure 1. Accelerated strength versus 7-day strength for method 1.

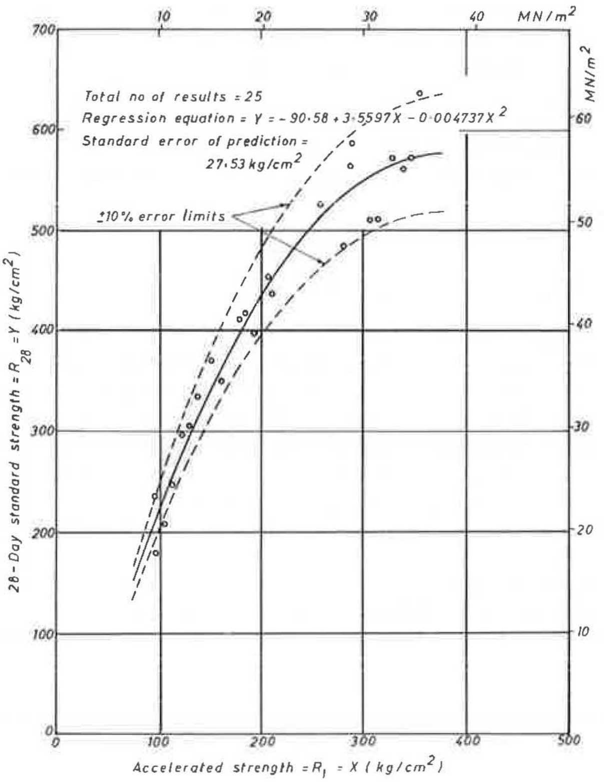


Figure 2. Accelerated strength versus 28-day strength for method 1.

Figure 3. Accelerated strength versus 7-day strength for method 2 and cements B and C.

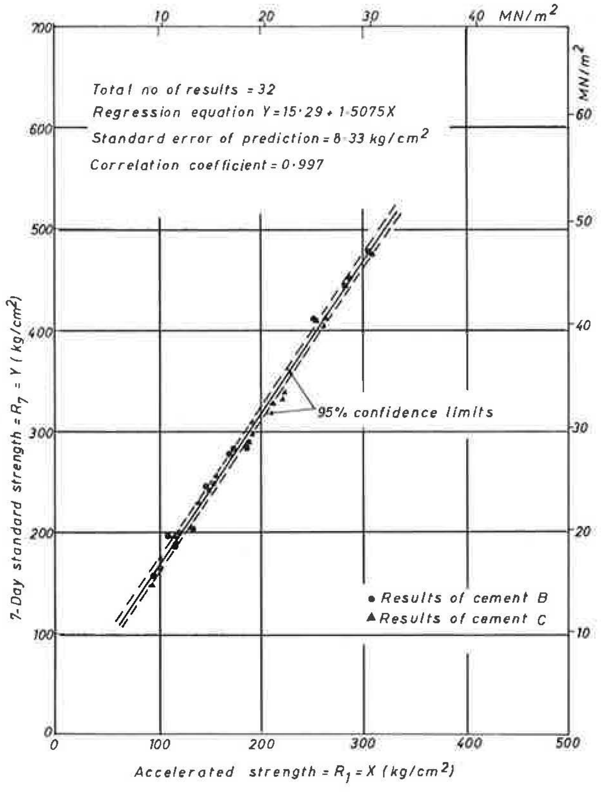
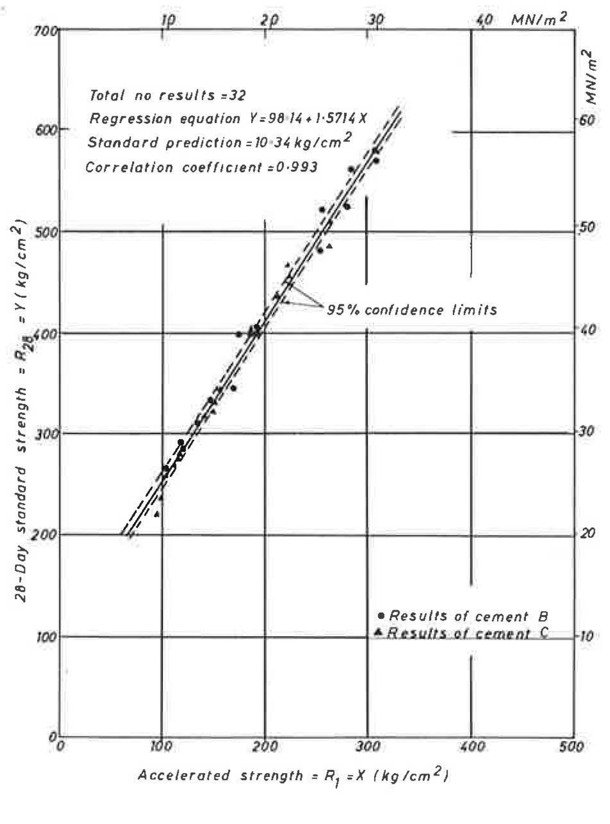


Figure 4. Accelerated strength versus 28-day strength for method 2 and cements B and C.



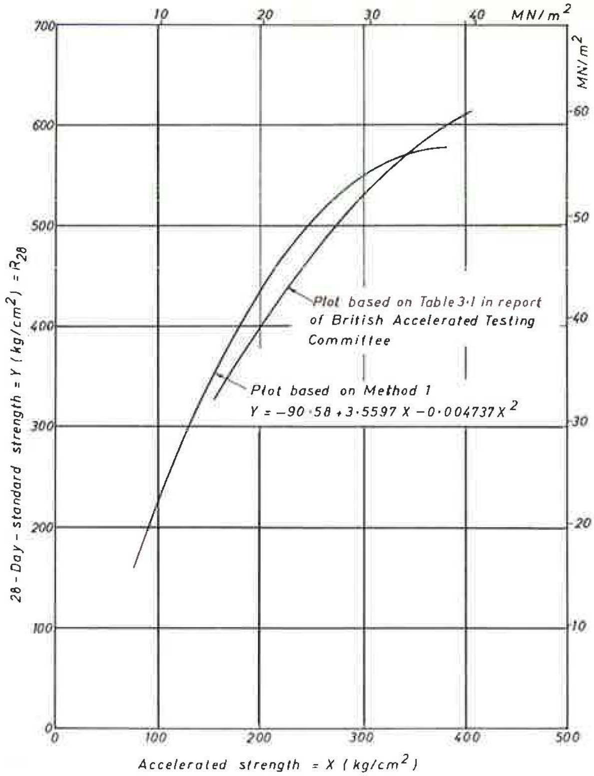


Figure 5. Method 1 versus British accelerated testing committee method.

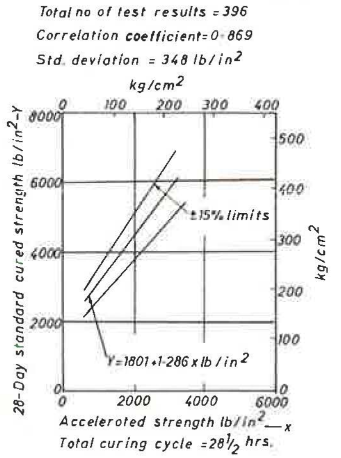


Figure 6. Accelerated strength versus 28-day strength.

Table 7. Correlation between the 7 and 28-day strengths of normally cured specimens.

Cement	Regression Equation	Standard Error of Prediction (kgf/cm^2)	Coefficient of Correlation
A	$R_{28} = 39.0 + 1.314R_7$	27.3	0.980
B	$R_{28} = 101.7 + 0.982R_7$	18.86	0.991
C	$R_{28} = 63.8 + 1.113R_7$	15.35	0.988
Total population for B and C	$R_{28} = 84.8 + 1.003R_7$	19.16	0.984

Note: 1 kgf/cm^2 = 0.098 MPa

MERIT OF ACCELERATED TESTING OVER CURRENT PRACTICE

In the absence of accelerated testing, current practice for estimation of 28-day strengths is based on the 7-day strengths actually obtained. To compare the relative merits of this method against the prediction of accelerated test method, linear correlations between R_7 and R_{28} for the different cements were determined. The results of the regression analysis are given in Table 7. Comparison of these with the results of accelerated test method 2 (Figure 4) shows that the latter are at least as reliable as the 7-day (normally cured) strength tests for prediction of the 28-day strengths and can be used confidently and advantageously in time.

CONCLUSIONS

1. Both the accelerated test methods tried appear to give satisfactory results for predicting the 28-day strength of concrete.
2. Method 2, a modification of the Canadian boiling method, is recommended over method 1 because it gives better accuracy in prediction and provides a simple linear relationship and because the operating procedure and curing characteristics are simpler than those of method 1.
3. The accelerated test method (method 2) can be advantageously adopted in place of 7-day (normally cured) strengths for reliable estimation of 28-day strengths.
4. The relationship obtained for prediction of 28-day strength appears to be independent of the concrete variables like cement content, water-cement ratio, and the source of cement.
5. The accelerated test method 2 can be further developed for wide application by using a large number of tests covering the practical range of concrete mixes used in the laboratory and field. This method can be used to help in the design of trial mixes, to evaluate the uniformity of field concrete for quality control, and to evaluate the strength and quality of concrete by replacing the 28-day strength test entirely.

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PRACTICAL APPLICATION OF MATURITY CONCEPT TO DETERMINE IN SITU STRENGTH OF CONCRETE

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Strength of field-cured concrete has been estimated by correlating the maturity to strength, the maturity being a function of age and curing temperature. Additional standard cylinders are cast, and the maturity of in-place concrete is determined from thermocouples cast into the concrete pour or maturity meters installed during placing of the concrete. Standard cylinder strength and field maturity are then correlated, and in-place strength is determined by using constants for the concrete used. To check the validity of this approach, a series of strength tests on 4-in. (10-cm) cubes at various maturities were conducted in the laboratory. These sets of cubes were cast from concrete obtained from various job sites in Toronto. Seven and 28-day cylinder strengths predicted from low-maturity cylinder strengths of the same set were evaluated. The constants of the strength-maturity equation were determined to suit the local normal concrete and to compare the results with those of Plowman. A satisfactory correlation was obtained. The method was used to predict the in situ strength of concrete slabs during construction of buildings at the University of Waterloo in 1971 and 1972. The contractor could then strip the forms early and determine if there were any inadequate curing conditions during winter construction. Tests were made during construction to check the actual strengths of the in situ concrete and to compare them with predicted theoretical values. The results were satisfactory. The strength gain of lightweight concrete floor slabs of a 37-storied tower recently completed in Toronto was monitored to determine the earliest time when posttensioning operation of the slabs could be carried out safely.

•THE standard method of determining the potential strength of concrete by using 6 by 12-in. (15 by 30-cm) cylinders cured in a fog room at 73.4 F (23 C) does not necessarily indicate the strength of the same batch of concrete in a structure. A number of people (1, 2, 3, 4, 5), mainly Plowman (3), have suggested methods of relating strength to maturity, maturity being a function of age and curing temperature. This approach to estimation of strength of a field-cured concrete has been successfully used on several occasions.

In practice, the method adopted involves the casting of additional standard cylinders and determination of the maturity of the in-place concrete. The latter is normally done by using thermocouples cast into the concrete pour or maturity meters installed during placing of the concrete. The data (standard-cured cylinder strength and field maturity) are then correlated, and the in-place strength is determined by using constants in the correlation equation for the particular type of concrete being used. Once a correlation between cylinder strength and the corresponding maturity is established, the strength of the structural element can be estimated from the maturity of the in situ concrete alone.

LABORATORY TESTS

To check the validity of this approach, limited test programs were carried out. The first involved the casting, each day, of a set of 4-in. (10-cm) cubes from one of the

sites on which routine testing was taking place. On each day, the site from which the samples were taken was chosen randomly; therefore, the specimens represented various suppliers and ingredients. A total of 22 sets of test specimens were cast. Out of each set of four cubes, two were cured at temperatures between 20 and 25 F (-6.67 and -3.89 C), and the others were cured in the standard fog room at 73.4 F (23 C). Individual cubes were tested at different ages to produce a range of maturities. The test results of individual cubes were then used to cross-predict the strengths of the other cubes in the same set but of different maturities by using Plowman's constants (3) for rapid-hardening cement. Constants derived from our observations based on the equation, percentage of strength of 28-day standard-cured concrete = A + B (log $M/10^3$), where constants A and B were used in the same way as Plowman's, and M is the maturity in deg F (C) hours.

The following values of the constants were used for calculating the percentage strength of 28-day standard-cured concrete at 65 F (18.33 C) in the case of Plowman's constants and 28 days at 73.4 F (23 C) in the case of constants derived from local tests:

Cement	From Plowman's Rapid-Hardening Cement	From Locally Conducted Tests
A	31.8	41.3
B	43.6	36.4

The evaluated data are shown in Figures 1, 2, and 3. One can see in Figures 1 and 2 that a good correlation exists between the predicted and actual strengths of cubes cured in the fog room. It was felt that, if the predicted strength based on the equation, percentage of strength of 28-day standard-cured concrete = A + B (log $M/10^3$), is within ±10 percent of the actual strength of the concrete, the method is acceptable. However, erratic results were obtained on the cubes cured at a low temperature [20 to 25 F (-6.67 to -3.89 C)] as can be seen in Figure 3. All the concrete tested was in the strength range of 3,500 to 5,000 psi (24 to 34 MPa) at 28 days.

Similar evaluations of 7 and 28-day, standard-cured, 6 by 12-in. (15 by 30-cm) cylinder strengths were carried out, and values were predicted from low-maturity cylinder strengths of the same set (generally 2 to 3-day, standard-cured cylinders). These data are shown in Figures 4, 5, 6, and 7.

Except for a few cases, the predicted strength is generally within ±10 percent of the actual strength of the cylinders and is generally the lower limit when it is outside the range. In other words, the strength predicted by the equation is lower than the actual strength, which, from a construction point of view, is safe and is a warrant for adequate precaution in matters such as proper curing condition, particularly during cold weather construction. Therefore, the relationship put forward by Plowman (3) is valid within reasonable limits for concrete used in the Toronto area provided the concrete is cured under normal conditions.

FIELD OBSERVATIONS

A limited field study was carried out at the Students Services and Administrative Studies Buildings, University of Waterloo, during the construction period in 1971. The tests were made to check the actual strengths of the in situ concrete and compare them with predicted strengths.

At the test sections of the concrete slab, thermocouples were installed, and extra cylinders were cast. At the same time 4-in.-diameter (10-cm) cylindrical tubes were installed in the slab adjacent to thermocouple locations. These tubes were constructed so that the concrete cast into them could be withdrawn on the day that the corresponding standard cylinder strength tests were made. The predicted strength from the maturity

Figure 1. Test results of 4-in. (10-cm) cubes based on Plowman's constants.

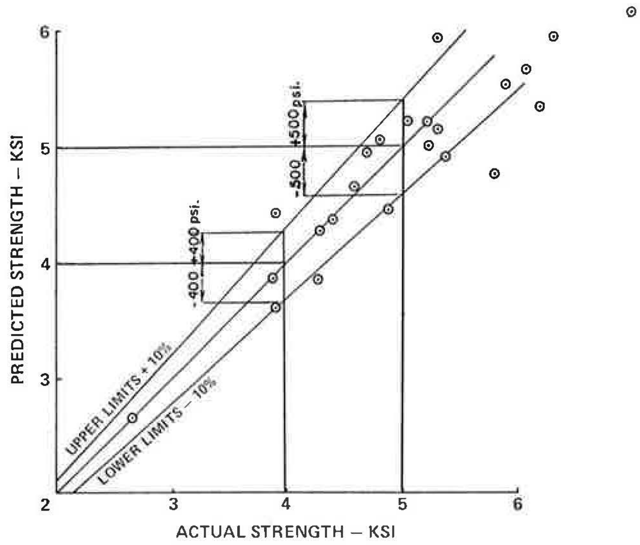


Figure 2. Test results of 4-in. (10-cm) cubes based on constants derived from local tests.

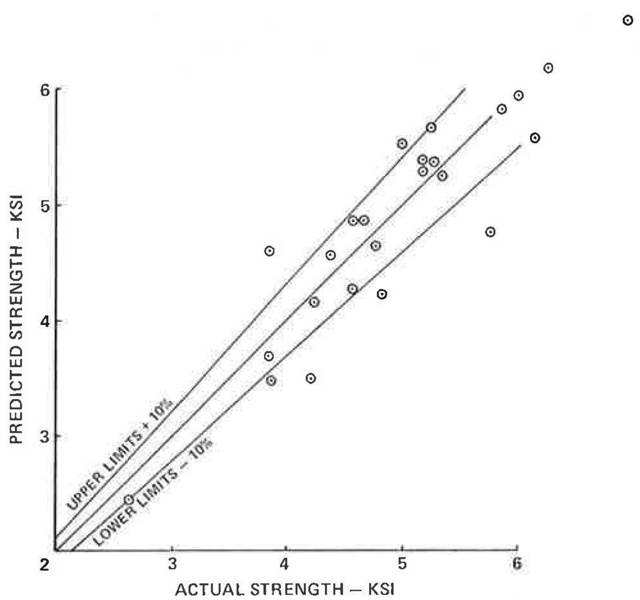


Figure 3. Test results of 4-in. (10-cm) cubes cured in freezing condition.

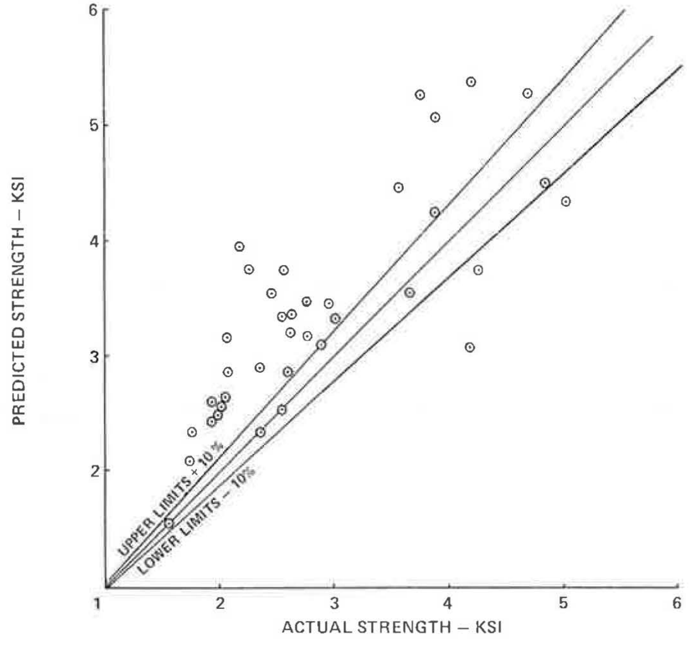


Figure 4. Tests made on 6 by 12-in. (15 by 30-cm) cylinders from Students Services Building and based on Plowman's constants.

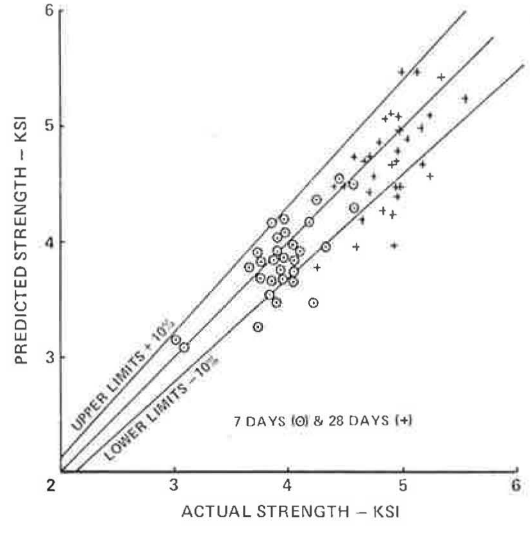


Figure 5. Tests made on 6 by 12-in. (15 by 30-cm) cylinders from Students Services Building and based on constants derived from local tests.

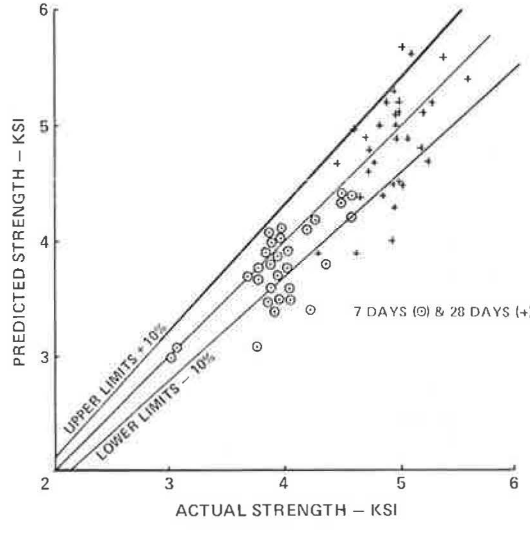


Figure 6. Tests made on 6 by 12-in. (15 by 30-cm) cylinders from Administrative Studies Building and based on Plowman's constants.

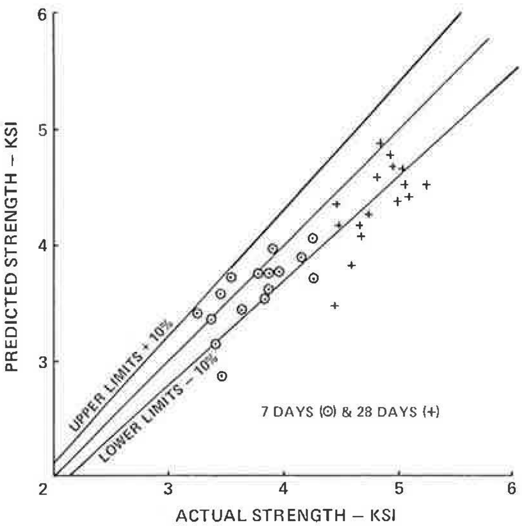


Figure 7. Tests made on 6 by 12-in. (15 by 30-cm) cylinders from Administrative Studies Building and based on constants derived from local tests.

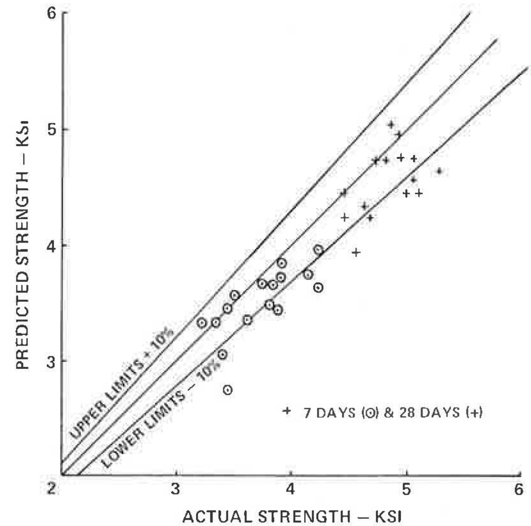
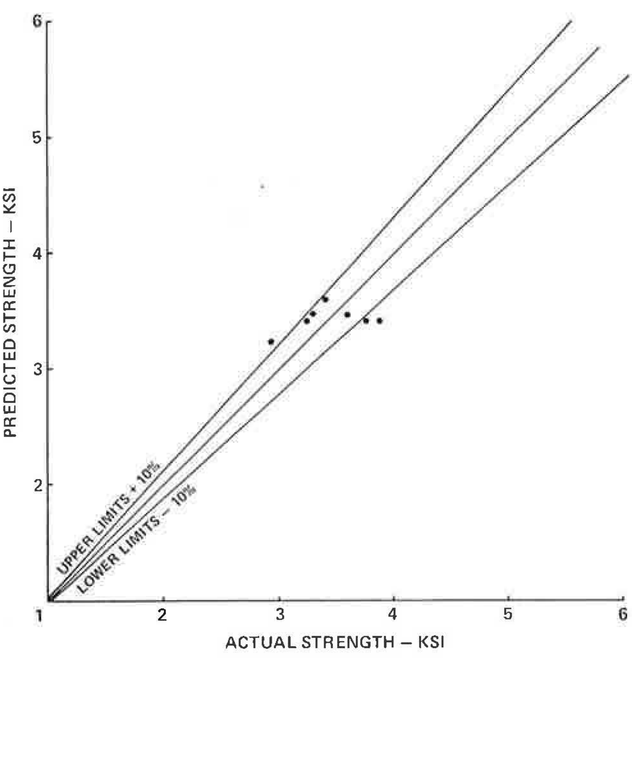


Figure 8. Tests on 4-in. (10-cm) push-out cores.



of the concrete and the actual strength of the push-out cylinder based on Plowman's equation are shown in Figure 8. In view of the small amount of test data and the small range of strength data, the correlation is not very clear. However, the predicted strengths of the push-out cylinders were close to the actual strengths.

This method indicates that the in situ strength of the concrete can be predicted with reasonable accuracy if the maturity of the field concrete and a relationship between the cylinder strength and the corresponding maturity are known. When the relationship between the maturity and the cylinder strength for a particular type of concrete is established, the monitoring of the field maturity alone could be repeatedly used on every floor of a building to help the contractor achieve early stripping of the forms or to indicate if there are any inadequate curing conditions during winter construction.

The strength gain of the lightweight concrete posttensioned floor slabs of the 37-storied tower recently completed by Direzione Lavori of Canada Limited in Toronto was monitored by maturity test. A requirement of a minimum concrete strength of 3,000 psi (20.7 MPa) was placed by the structural engineer before the posttensioning operation and the stripping of the form work could be carried out safely. The maturity of the concrete was checked every 24 hours, and immediate calculation of the in situ strength was made. This helped the contractor to achieve a faster rate of construction.

CONCLUSIONS

Although a considerable amount of research in this field is needed to solve many unanswered questions, it is felt that this approach is a valuable means, from the economic as well as safety point of view, of estimating early strength gain of in situ concrete.

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PERMEABILITY TESTING OF PLASTIC CONCRETE

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Conventional permeability tests on hardened concrete suffer some limitations such as uncertainty of the physics governing the flow. These tests are painstaking and expensive and yield inconsistent results. Testing concrete at the plastic stage is proposed as a way of rationalizing and accelerating the test. As a first step toward full realization of this concept, the permeability characteristics of retarded plastic concrete have been investigated by using a specially designed cell and a falling-head permeameter. During this investigation, particular attention was paid to (a) elimination of the effect of boundary flow on the true values of the coefficient of permeability of the permeate; (b) verification of the validity of Darcy's law for flow-through plastic concrete; and (c) evaluation of the effects of variation in water-cement ratios and the hydration process on values of the coefficient of permeability of the permeate. Results so far available confirm that the Darcy flow is applicable to plastic concrete. The Darcy coefficient of permeability of the mixes used in this project is generally of the order of 10^{-5} cm/s and increases exponentially as the water-cement ratio increases. The effect of addition of sugar, as a retarder, is to reduce values of the coefficient of permeability of the permeate. Similarly, hydration reduces the coefficient of permeability of the permeate, which decreases with time.

•THE determination of the coefficient of water permeability of hardened concrete involves complicated and expensive test preparation, and the test itself is time consuming. In addition, there appears to be no consensus of opinion on the physics governing the migration of water through hardened concrete. Flow of water through hardened concrete has been variously conceived. Hughes (1) and Murata (2, 3, 4, 5), for example, favor an absorption process, and others conceive the problem as a flow process conforming to Darcy's law. The above limitations are further compounded by the large discrepancy that can occur between permeability test results for apparently identical specimens.

I have postulated that a departure from the convention, by carrying out permeability tests on the fresh mix rather than on the hardened concrete, would, if successfully developed, overcome some of the previously mentioned weaknesses of the conventional test. From theoretical considerations, this unconventional approach should overcome the fundamental issue regarding the pertinent physics governing flow through the material. Plastic concrete has a structure that is similar to a cohesive-frictional soil (6), and the movement of water through such porous media is a flow process.

From a methodological standpoint, the proposal will yield savings in both time and effort. When combined with the fact that the proposed test has to be undertaken on a freshly mixed concrete, the new test qualifies as an accelerated form of permeability testing on concrete. Full realization of this concept requires the establishment of either or both of the following:

1. A correlation between the permeability of plastic concrete with that of the hardened concrete; or
2. A correlation between the permeability of plastic concrete with relevant performance characteristics of concrete, e.g., durability and development of interstitial pressures in dams.

However, the prerequisite to these steps must be a thorough investigation of the methodology of permeability testing of plastic concrete and the permeability characteristics of the material itself. When these have been established, the vital, but secondary, stage of correlating the relevant parameters can then proceed.

In this paper, an investigation into the permeability of plastic concrete is discussed that is aimed at studying the permeability characteristics of concrete mixes at zero time, i.e., just after mixing before any significant setting of the mix takes place. So that this can be achieved, the mixes have been retarded by addition of a small dose of sugar. Particular attention has been paid to the common problem of the boundary flow that occurs during permeability tests. The effect of changes in the water-cement ratio, which is the most crucial of concrete mix variables, and the effect of setting due to cement hydration on the permeability values have also been investigated.

The following notation is used in the paper:

A = cross-sectional area of permeate,
A_c = cross-sectional area of the core of the permeate,
A_p = cross-sectional area of the periphery of the permeate,
a = cross-sectional area of standpipe or manometer tube in the permeameter,
B = constant,
C = constant,
H = height of water in the standpipe above the top surface of the permeate at any time,
H_o = initial height of water in the standpipe above the top surface of the permeate,
H_t = final height of water in the standpipe above the top surface of the permeate,
dH = drop in height of water in the standpipe in time dt,
i = hydraulic gradient,
k = coefficient of permeability of the permeate,
k_c = coefficient of permeability of core of the permeate,
k_p = coefficient of permeability of periphery of the permeate,
L = thickness of the permeate,
Q = quantity of flow through the permeate,
Q_p = quantity of flow through periphery of permeate,
Q_c = quantity of flow through core of permeate,
$\beta = Q_p/Q_c$,
t = time interval, and
v = discharge velocity.

THEORETICAL BASIS FOR PERMEABILITY CELL DESIGN

Pilot tests showed that the boundary flow between the mix and the wall of the cell was significant despite various contrivances that were devised to arrest it. It was therefore decided to tolerate boundary flow and then to modify both the theory and design details of the permeability cell accordingly. Two flow regimes were distinguished (Figure 1): (a) core flow, from which the true k-values were computed, and (b) periphery flow, which included boundary flow. The final design of the cell achieved isolation of these two regimes.

The fundamental law defining macroscopic permeation through a porous medium was established by Darcy in 1856. The law states that

$$V = ki \tag{1}$$

Equation 1 may be rewritten as

$$Q = Aki \tag{2}$$

The falling-head permeability test is usually adopted for materials whose k-values are anticipated to fall below 10^{-1} cm/s (7). On this basis, equation 2 has been developed to simulate the design features of the cell. The basic theoretical approach is available elsewhere (8).

If the level of water in the standpipe fitted above the cell falls by dH in time dt, then

$$\int \frac{H}{L} dt(A_p k_p + A_c k_c) = -a \int dH \tag{3}$$

In any interval of time,

$$\frac{Q_p}{Q_c} = \frac{A_p k_p}{A_c k_c} \tag{4}$$

Substituting equation 4 into equation 3 and evaluating and rearranging the integrals give the true permeability of the material:

$$k_c = \frac{2.3\ aL}{A_c\, t(1 + \beta)} \log_{10} \left(\frac{H_o}{H_t}\right) \tag{5}$$

Every term on the right side of equation 5 can easily be determined, except the effective core area A_c. It should be emphasized that this is not equal to the area bound by the imaginary demarcation lines, i.e., the broken vertical lines in Figure 1. The effective core area is the area at the top of the concrete specimen from which the quantity of water collected via the central outlet has actually emanated. This is better explained by reference to flow nets, the principles of which are outlined by Harr (9). The effective core area is shown at the top of Figure 2; the flow channels from this area cover the core area at the base.

The flow-net technique was used to determine A_c. Based on the observation that each flow channel conveys the same quantity of water, the ratio of the number of channels terminating over the core zone at the base to the total number of flow channels gives the ratio of core flow to the total flow. The effective core area contributing this proportion of flow is then computed from the flow-net analysis shown in Figure 2. Several flow nets were drawn, and an empirical relationship between quantity ratio and area ratio was established.

Boundary conditions can also be defined from theoretical considerations and for the particular cell used in this project. They are as follows:

1. The minimum flow ratio is zero, and the corresponding area ratio is zero.
2. The maximum flow ratio is 0.36, and the corresponding area ratio is 0.36.

Data obtained from the flow-net analysis are plotted with the boundary values in Figure 3. Nonlinear regression analyses were carried out in an attempt to fit either a polynomial or an exponential curve to all the points, but the outcome seemed improbable. However, if the upper boundary value was discarded, albeit theoretically plausible, a linear correlation became apparent. The theoretical points, i.e., the upper and lower bound values, were next ignored, and a regression line was fitted to the flow-net points only. The correlation coefficient for this regression line is 0.99.

Given any value of flow ratio determined from tests based on this particular cell, the corresponding area ratio can then be interpolated from the correlation. The effective core area A_c can therefore be calculated and then substituted into equation 5.

Figure 1. Flow regimes.

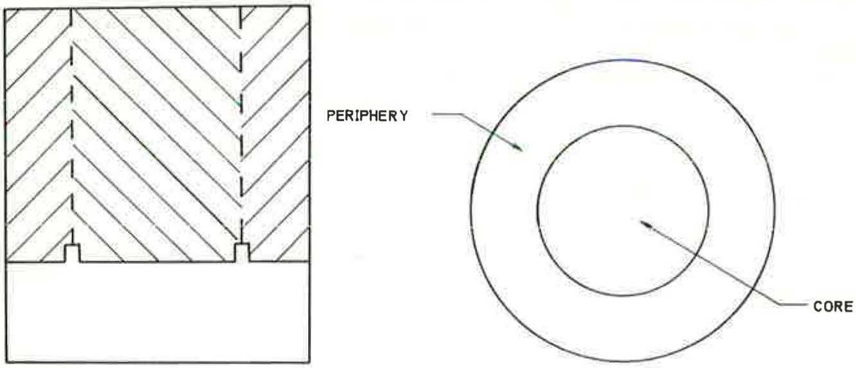


Figure 2. Flow net for test systems.

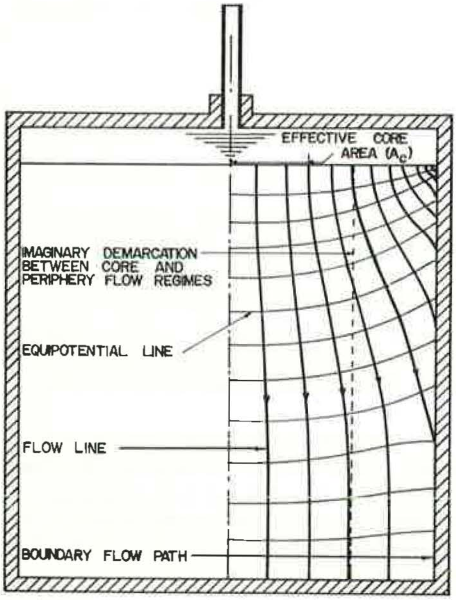
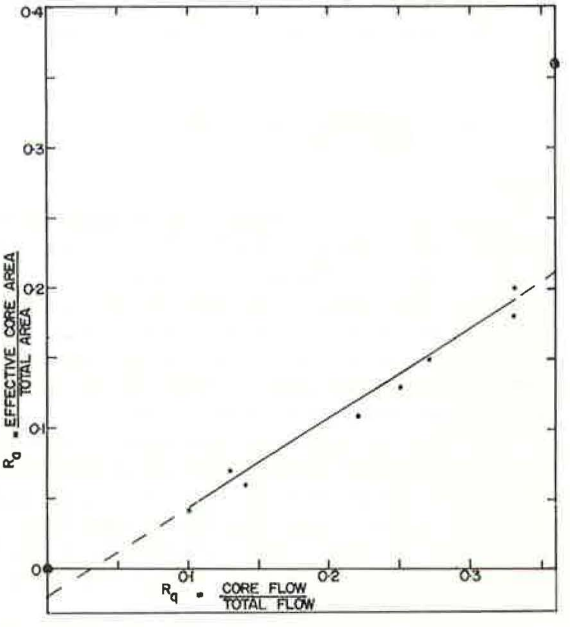


Figure 3. Correlation between flow ratio and area ratio.



APPARATUS

Permeability Cell

Details of the final design are shown in Figure 4. The cell was constructed from cast aluminum to limit rusting. The contact time between each mix and the cell was short, and, consequently, the effect of any reaction between the concrete and the aluminum was considered to be negligible. The three cell components, top, body, and base, are held together with brass screws. Rubber O-rings that are housed in grooves are used to prevent leakage at the joints.

Inlet and bleed valves are provided at the top, and the inside of the upper face is domed to facilitate air bleeding. Perforated aluminum and porous plastic (Vyon) discs sit on a recess at the upper end of the body. The porous plastic prevents any possible upward migration of concrete fines, while the aluminum enhances the flexural rigidity of the porous plastic. The inside wall of the body is machined to produce a continuous fine spiral groove from top to bottom; this aims at improving the keying of the mix to the cell and lengthening the boundary flow path. The combined effect was to minimize the severity of boundary flow. The base has a central depression, 44.5 mm in diameter and 3.2 mm deep, and a central tapping leading to an outlet valve. An annular ridge, 3.2 mm wide and 3.2 mm deep, separates the central depression from an annular groove that collects the peripheral flow.

Permeameter

The permeameter, shown in Figure 5, is similar in principle to conventional falling-head permeameters. However, the design satisfies two specific requirements:

1. Reversible flow can be obtained; therefore, both upward and downward flow tests can be done.
2. A wide range of initial hydraulic heads can also be obtained. High pressures are achieved by coupling to a standard Bishop pressure apparatus (10).

TEST DETAILS

Specification of Concrete Mix and Materials

Budgetary constraints limited the number of mix variables to be investigated to one: the water-cement ratio, which is the most crucial to the characteristics of concrete. The water-cement ratios were chosen to give the following mix consistencies:

Compacting Factors	Water-Cement Ratio
0.78	0.40
0.85	0.43
0.92	0.47
0.95	0.50

Ordinary portland cement and river sand and gravel were used. The aggregate was separated into single sizes before it was recombined to exactly produce grading curve (11). The laboratory air-dried aggregate was soaked for 24 hours before mixing. The total water-cement ratio mentioned earlier included the moisture absorbed by the

Figure 4. Permeability cell.

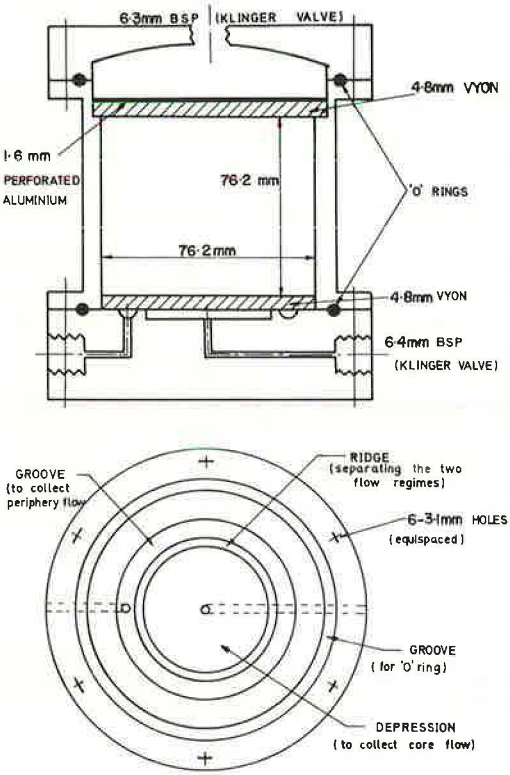


Figure 5. Permeability test equipment.

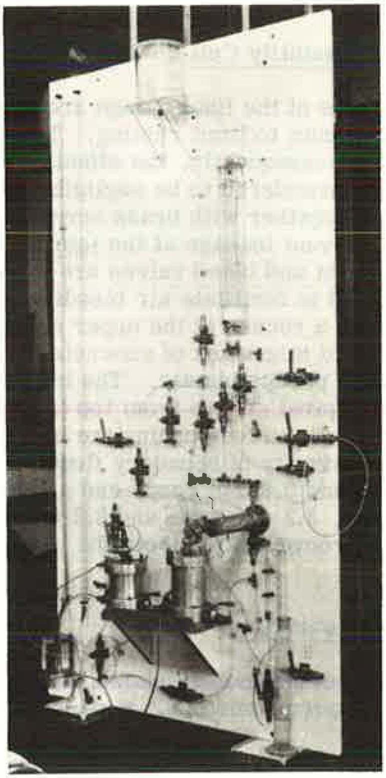
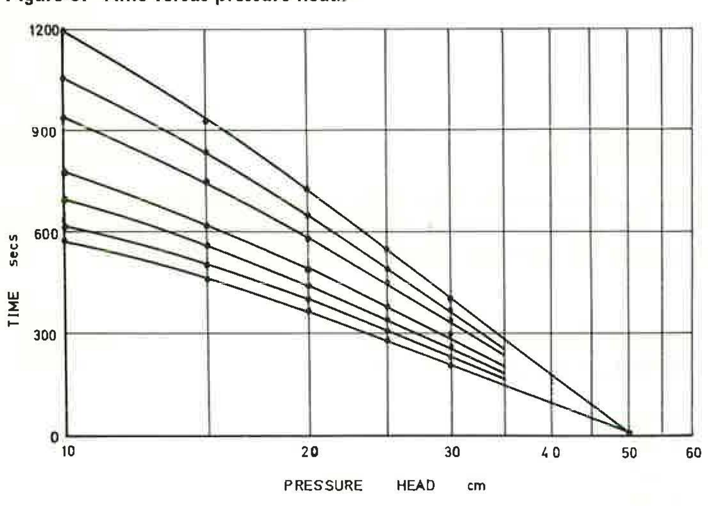


Figure 6. Time versus pressure head.



aggregate. The specific gravity and absorption of the graded material are 2.5 and 1.4 percent respectively. An aggregate-cement ratio of 4.5 was selected because it was within practical limits.

A small quantity of sugar was added to some of the mixes to retard cement hydration. A sugar-cement ratio of 0.0025 by weight was adopted. This has been shown to sufficiently retard strength gain within 24 hours (12).

Test Procedure

A predetermined quantity of the mix was vibrated into the cell. Pilot tests had shown that vibration was the most effective practical technique for compacting the test specimen to minimize boundary flow. The specimen was then de-aired by applying a low back pressure that caused a gradual upward flow of de-aired water through the specimen. When no further air was observed from the top valve, the back pressure was cut off, and the specimen was ready for test.

The permeability test commenced exactly 1 hour after mixing; this time had earlier been found to be just sufficient for careful sample preparation. The test essentially consisted of recording the drops in the hydraulic head, the corresponding time intervals, and the quantities of flow from both the core and periphery zones.

Except for the investigation of time effects (or hydration), each sample was used for only one test. Six tests were made for each mix (i.e., water-cement ratio).

DISCUSSION OF RESULTS

Validity of Darcy's Law

It was postulated, as a premise for the laboratory study, that flow through plastic concrete obeys Darcy's law. The empirical data may be used to verify this postulation.

For each test, all terms in equation 5, except t and H_t, are constants. The equation may therefore be written in the following form:

$$t = B - C \log_{10} H_t \tag{6}$$

As equation 5 was developed from Darcy's law, the flow of water through plastic concrete actually obeys Darcy's law if a plot of t-values against $\log_{10} H_t$ is linear as implied by equation 6. Such plots were made from the test data, and Figure 6 is typical. The various curves are for different ages of the mix and increase upwards at 30, 60, 90, 120, 180, 240, and 300 min from the end of mixing.

Down to the 20.0-cm head, the flow generally obeys Darcy's law but deviates at lower gradients where nonlinearity occurs. This partial non-Darcy flow is attributed to particle migration that occurs with the progress of flow. Incomplete saturation as suggested by Matyas (13) and boundary flow as suggested by Mitchell and Younger (14) can be discounted as care was taken to eliminate these. Moreover, if incomplete saturation had been responsible, deviation would have occurred right at the outset rather than at lower heads since the degree of saturation would improve as the permeation front progresses. The establishment of particle migration in clays (14) and the observation that the head at which deviation occurred varied with the initial head lend support to the migration hypothesis.

Variation of k With Water-Cement Ratio

The permeability of plastic concrete obtained from this study is generally of the order

of 10^{-5} cm/s. The results are shown in Figure 7, and the average values of the permeability are given in the following table:

Water-Cement Ratio	Equivalent Moisture Content (percent)	k-Values (10^{-5} cm/s)
0.50	9.10	7.76
0.47	8.50	7.52
0.43	7.72	6.32
0.40	7.27	2.71

Figure 7 suggests that k increases exponentially as the water-cement ratio increases. This result is considered logical because increasing water-cement ratio increases the porosity of the mix. The result is also consistent with the trends obtained for hardened concrete (15, 16). For moisture contents of 2 percent above optimum and more, the k-values for soils (soil being similar in structure to plastic concrete) also increase with moisture content (17).

I had previously determined indirect k-values of the order of 10^{-8} cm/s for plastic concrete from one-dimensional consolidation tests (18). The high consolidation pressures have the effect of considerably reducing the void ratio of the test specimen and consequently result in very low k-values. In this light, the two sets of results are considered to be compatible, and this order of difference is not uncommon in soil mechanics.

Effect of Sugar

The average of six values obtained for an unretarded mix (i.e., without sugar), having a water-cement ratio of 0.50, is 1.9×10^{-4} cm/s. When the unretarded mix was compared with the corresponding mix containing sugar, it was seen that the addition of sugar reduced the permeability of the mix.

The action of organic materials, such as sugar, in retarding the setting of cement is not yet fully understood. However, it is thought that some retard by adsorption through either their carboxyl or hydroxl groups, and such actions occur rapidly initially (19). The adsorption process would result in the fixation of part of the pore water and thereby reduce the effective pore channel (20) and give a less permeable structure.

Effect of Time

The well-known theories of Le Chatelier and Lea suggest that an intergrowth of crystals lock together during the hydration process and thus result in a simultaneous diminution in pore size and a change of pore geometry. This continuous change in pore structure would cause k-values to decrease with time, as shown in Figure 8.

The shape of the curve suggests that the k-value may approach a stable value. With further investigation into the effect of time, including hardened concrete, it may be possible to evolve a predictive correlation between the k-value and time. In this case, the k-value for hardened concrete can then be predicted from permeability tests on the fresh mix.

CONCLUSIONS

Experience gained from the present project confirms that permeability testing of plastic concrete is a feasible and viable proposition. However, a more comprehensive investigation into the permeability of plastic concrete is necessary before the program

Figure 7. Variation of coefficient of permeability with moisture content.

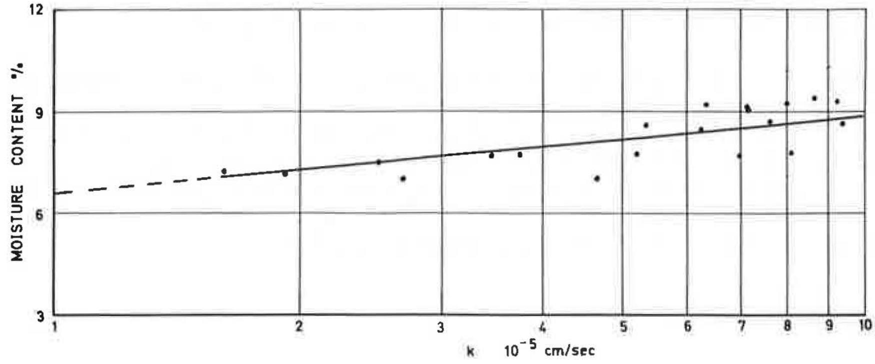
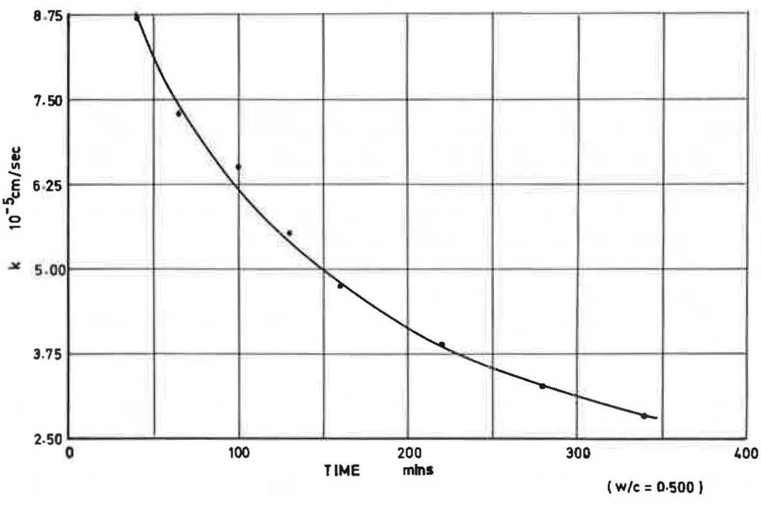


Figure 8. Variation of coefficient of permeability with time.



is extended to include hardened material. The rate at which the development proceeds will be expedited if several researchers cooperate.

The following conclusions have been made from the results of this study:

1. Flow of water through plastic concrete obeys Darcy's law down to a limited hydraulic gradient that varies with the initial value;
2. Non-Darcy flow at the lower gradients seems to be due to particle migration;
3. k-values increase exponentially with increasing water-cement ratio;
4. Retardation, by addition of sugar, reduces the permeability of plastic concrete; and
5. The effect of hydration causes the reduction of k with time.

ACKNOWLEDGMENTS

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EVALUATION OF A CHEMICAL TECHNIQUE TO DETERMINE WATER AND CEMENT CONTENT OF FRESH CONCRETE

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This report presents information obtained from an evaluation of a chemical procedure for determining the water and cement content of a concrete in the plastic state. The procedure uses chloride ion titration to determine water content and flame photometry (calcium signature) to determine cement content. This study evaluated the procedure to determine if it could be used to estimate concrete strength potential and to define to what extent test results are influenced by aggregate type, aggregate moisture conditions, aggregate absorption capacity, concrete mix proportions, mix time, and time of sampling. The fieldworthiness of the system was also evaluated. Results indicate that the procedure can rapidly (approximately 15 min) determine the water and cement content of fresh concrete and that it can be used to predict strength potential with an accuracy equal to that of predicting strength from known mix proportions. Aggregate type was the only major concrete parameter that significantly influenced test results. Although aggregate moisture condition, mix proportions, and length of mixing time also influenced test results, their influences were minor. The field tests have indicated that the system is fieldworthy and mobile.

•INSPECTION and testing procedures currently being used to determine the quality of concrete involve a time lag between concrete placement and the evaluation of concrete quality (compression or beam tests). In addition, the current tests do not relate directly to either the material or the construction parameters that influence concrete quality.

This study evaluated the potential of a chemical technique originally developed by Kelly and Vail of the Greater London Council for rapidly determining the water and cement content of fresh concrete (1). The study determines if the procedure can be used to estimate concrete strength potential and defines to what extent test results are influenced by aggregate type, aggregate moisture conditions, aggregate absorption capacity, concrete mix proportions, mix time, and time of sampling. The fieldworthiness of the system was also evaluated.

PROCEDURE FOR DETERMINING WATER AND CEMENT CONTENT

The selection of analytical techniques for determining water and cement content was based on the criterion that the test should be rapid (<15 min), cheap, fieldworthy, and safe.

Water Content Determination

The method for water content determination is based on the theory that water in fresh concrete is available for intermixing with aqueous solutions. Thus, if an aqueous solution is of known strength and is not absorbed by the aggregate or the cement, the volume of water in a concrete sample can be determined analytically by determining the

concentration of the intermixed solution. That is, if A is the volume of water in the mix, B and S_1 are the volume and strength respectively of the aqueous solution, and S_2 is the strength of the intermixed solution, then

$$B \times S_1 = (A + B)S_2 \quad (1)$$

From equation 1, A can be calculated if B and S_1 are fixed and S_2 is measured. To measure the strength of the intermixed solution, the Volhard back-titration method is used with sodium chloride as the solute. When the concrete contains chloride from other sources, the procedure requires the use of both a sample and a blank. The Volhard back-titration method, with its white to reddish-brown end point, has the advantage of being accurate, rapid (average time required 7 min 30 s), and simple enough for use by persons without analytical experience.

Figure 1 shows the equipment required for determination of water content. The equipment consists of a mechanical shaker; two wide-mouthed plastic bottles; 10-ml, 5-ml, 2.5-ml, and 2-ml constant-volume dispensers; two 50-ml and one 10-ml automatic pipettes; one 100-ml burette; two 50-ml volumetric pipettes; two 500-ml volumetric flasks; and two 500-ml Erlenmeyer flasks.

The procedure for water determination is as follows:

1. Weigh out two separate 1-kg samples of concrete and place each sample in a wide-mouthed bottle. Add 500 ml of 0.5 N sodium chloride solution to one bottle (sample) and 500 ml of distilled water to the other bottle (blank).
2. Seal the bottles and place them in a mechanical shaker; operate it for 3 min.
3. Remove the bottles from the shaker and allow the contents to settle for 3 min.
4. Pipette 50-ml samples of clear supernatant liquid from the sample and blank bottles and add them to separate Erlenmeyer flasks. To each flask (sample and blank) add 10 ml of 50 percent nitric acid, 2 ml of nitrobenzene, and 5 ml of ferric alum; shake them well.
5. Determine the chloride content of the sample and blank flasks by adding excess silver nitrate (50 ml of 0.5 N $AgNO_3$ for sample and 10 ml of 0.5 N $AgNO_3$ for blank) and by back-titrating with 0.05 N potassium thiocyanate (Volhard back-titration).
6. Record the quantity of potassium thiocyanate required to reach the white to reddish-brown end point in both the sample and the blank. Use Figure 2 to determine the water content of the mix. [The quantity of KCNS (ml) required for sample titration plus the back-titration of the blank (100 minus the KCNS required for blank titration) equals the abscissa of Figure 2.]

Cement Content Determination

The cement determination technique is based on the assumptions that (a) cement can be dispersed in water and held uniformly in suspension so that a representative sample can be obtained; (b) a quantitative solution of the cement in nitric acid can be achieved by adding cement to the acid while it is rapidly stirred without external heat; and (c) that calcium can be determined by a flame photometer in relatively high concentrations in the nitric acid solutions without prior removal of silica and the sesquioxides.

Figure 1 shows the equipment required for the cement tests. The apparatus for preparing and sampling the cement-water suspension consists of a nest of sieves (No. 4 and No. 50) over a side-agitator domestic washing machine and three automatic pipettes. One pipette collects the constant volume cement-water sample from the washing machine; the others dilute the sample with nitric acid and water. An ordinary domestic high-speed stirrer (milk-shake type) provides agitation for dissolving the cement suspended in the acid solution. A flame photometer is used to determine the calcium (cement) concentration.

Briefly, the major steps for cement determination are as follows:

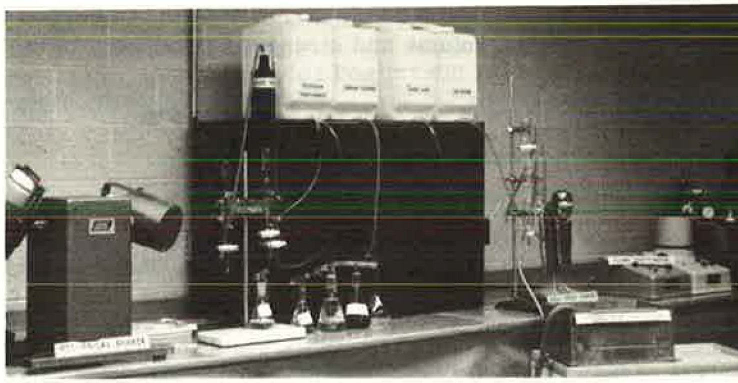


Figure 1. Equipment used in the Kelly-Vail procedure for determining the water and cement content of fresh concrete.

Figure 2. Results of water analysis.

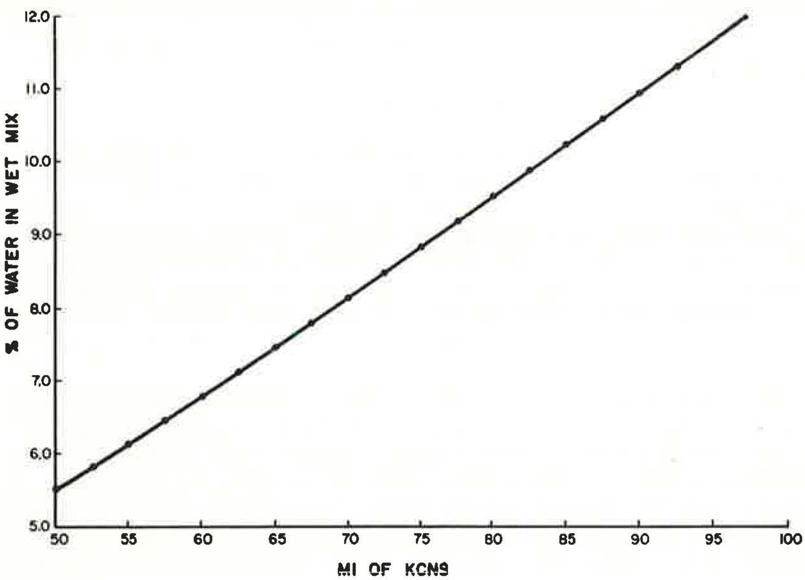
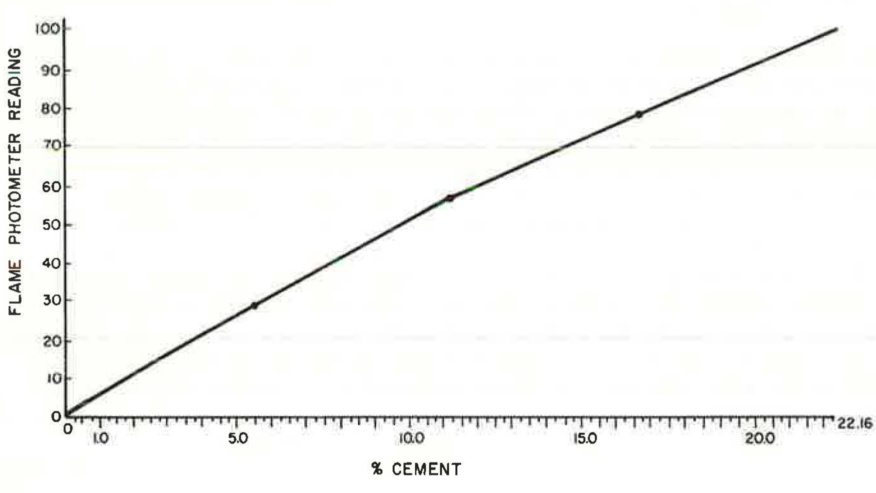


Figure 3. Results of cement analysis.



1. Fill the washing machine with 37.8 liters of tap water; place nest of sieves over the machine; start the agitator, and pump it to recirculate water.
2. Place a 1-kg concrete sample on the nest of sieves, and wash the cement from aggregate particles with the recirculating hose.
3. Allow agitation-recirculation operation to continue for 3 min. Attach the small hose to the automatic pipettes, and then clamp the recirculating hose nozzle so that the cement suspension will flow through the small hose and fill the automatic pipette (125 ml).
4. Empty the sample of cement suspension into a mixing cup, and wash down the pipette with 10 ml of 5 percent nitric acid from the upper pipette. Concurrently, dilute the acid-cement solution with 300 ml of tap water from the third pipette.
5. Stir the contents of mixing cup on high-speed mixer for 3 min.
6. Calibrate the flame photometer with a calcium standard and measure the calcium content of solution in the mixing cup. See Figure 3 for converting the readout to the cement content. (The calcium standard is prepared to equal 1.5 g/liter of cement, approximately 0.94 g/liter of $CaCO_3$.) The average time for a cement determination by an experienced operator is 7 min 10 s.

LABORATORY AND FIELD TESTS

Laboratory Tests

The laboratory test series evaluated three aggregate combinations, three mix proportions, two mix times, and two aggregate moisture conditions. The three aggregate combinations were Maryland quartz (coarse and fine), sand and gravel, and sand and crushed limestone (Figure 4). The mix proportions (Table 1) represented approximately 3,000, 4,500, and 6,000-psi (20.7, 31.0, and 41.4-MPa) concretes. A standard mix time of 5 min was used for each of the three mixes, and a second 4,500-psi (31.0-MPa) mix was tested by using a 45-min mix time. The two aggregate moisture conditions were air dried and saturated with some surface moisture.

Batches of 2 ft^3 (0.06 m^3) were used for all the series of tests. This was sufficient for a slump test and six 6 by 12-in. (15 by 30-cm) cylinders, in addition to the two 10-lb (4.54-kg) samples used for the water-cement analysis.

A complete standard water-cement analysis was run on both samples. The companion 6 by 12-in. (15 by 30-cm) cylinders were moist cured, three were broken at 7 days, and three were broken at 28 days.

Field Tests

Field tests were conducted at two construction sites by evaluating the mobility, reliability, and field worthiness of the system.

The test equipment was transported in a ready-to-use configuration in a pickup truck with a camper shell (Figure 5). To be operational, the self-contained unit requires only water from an external source.

The field tests evaluated ready-mix delivered concrete of three aggregate combinations and three mix designs. The aggregate combinations were lightweight coarse aggregate and sand, siliceous gravel and sand, and calcareous gravel and sand. The mix designs represented a 3,500-psi (24.1-MPa) structural lightweight concrete and a 4,500 and 3,000-psi (31.0 and 20.7-MPa) normal-weight concrete. [The actual batch proportions were not checked for the 3,500 and 4,500-psi (24.1 and 31.0-MPa) mixes because the batch plant was remote from the construction site and the water-cement test setup.]

The test procedure consisted of obtaining a water-cement content test sample from the same concrete that was used to prepare standard quality control cylinders. A complete water-cement analysis was run on all samples.

Figure 4. Aggregate gradations used in concrete tests.

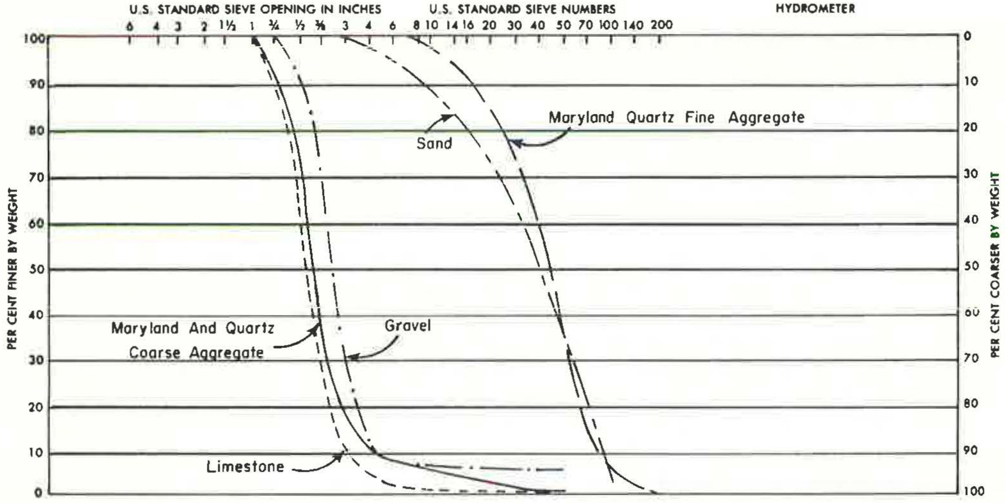


Table 1. Analysis of laboratory mixes.

Batch No.	Aggregate Type[a]	Coarse Aggregate (percent) Moisture	Coarse Aggregate (percent) Mix Proportion	Fine Aggregate (percent) Moisture	Fine Aggregate (percent) Mix Proportion	Cement Mix Proportion (percent)	Water (percent) Free	Water (percent) Total	Water-Cement Ratio[b]	Strength (psi) 7-Day Avg	Strength (psi) 28-Day Avg	Slump (in.)	Mix Time (min)
1	MdQ	0.56	48.5	7.86	24.6	18.40	8.39	8.73	0.45	3,600	4,785	–	5
2	MdQ	0:96	49.0	12.01	31.4	14.24	8.80	9.18	0.62	2,590	3,585	–	5
3	MdQ	0.41	49.3	15.38	34.4	11.71	8.85	9.25	0.75	1,470	2,420	–	5
4	MdQ	0.44	49.0	16.74	32.6	14.24	8.64	9.02	0.61	2,640	3,910	–	45
5	L-S	1.30	41.8	6.20	31.4	19.55	8.74	9.62	0.45	5,258	6,460	–	5
6	L-S	1.40	42.1	6.30	36.0	15.14	8.54	9.47	0.56	4,250	5,494	8.0	5
7	L-S	1.20	42.4	6.40	38.4	12.57	8.53	9.49	0.68	2,700	4,061	–	5
8	L-S	1.30	42.1	5.70	35.9	15.14	8.42	9.35	0.56	4,062	5,382	5.5	45
9	L-S	0.10	41.8	0.40	29.9	19.55	7.91	8.80	0.40	5,612	6,910	6.5	5
10	L-S	0.09	42.1	0.40	34.0	15.14	8.02	8.95	0.53	4,310	5,335	8.5	5
11	L-S	0.10	42.4	0.50	36.2	12.57	8.08	9.02	0.64	3,024	4,085	8.0	5
12	L-S	0.10	42.1	0.40	34.0	15.14	8.02	8.96	0.53	4,062	5,215	3.0	45
13	MdQ	0.07	49.0	0.15	28.5	14.24	7.89	8.28	0.55	3,431	4,290	4.5	5
14	MdQ	0.05	49.0	0.15	28.5	14.24	7.88	8.27	0.55	3,349	4,005	1.5	45
15	G-S	3.12	34.6	3.67	33.2	23.70	9.00	10.59	0.38	6,550	7,460	6.0	5
16	G-S	3.32	35.0	3.69	39.8	16.84	9.12	10.80	0.54	4,186	5,390	9.0	5
17	G-S	3.33	35.2	3.00	42.9	13.49	9.04	10.76	0.67	2,730	3,930	9.5	5
18	G-S	3.20	34.9	4.26	40.4	16.79	8.97	10.65	0.53	4,304	5,220	4.0	45
19	G-S	0.36	34.6	0.22	32.3	23.62	7.97	9.61	0.34	6,733	7,670	1.0	5
20	G-S	0.32	34.9	0.21	38.8	16.79	7.98	9.69	0.48	4,710	5,750	4.5	5
21	G-S	0.32	35.2	0.19	41.7	13.48	8.01	9.77	0.59	3,215	4,140	3.5	5
22	G-S	0.27	34.9	0.21	38.8	16.79	7.96	9.68	0.47	4,740	5,770	1.0	45

Note: 1 psi = 6.9 kPa. 1 in. = 2.5 cm.

[a]MdQ = Maryland quartz coarse (absorption capability = 0.35 percent) and Maryland quartz fine (absorption capability = 0.75 percent). L-S = crushed limestone coarse (absorption capability = 1.30 percent) and river sand fine (absorption capability = 1.15 percent). G-S = gravel coarse (absorption capability = 3.65 percent) and river sand fine (absorption capability = 1.15 percent).
[b]Based on free-water content.

Figure 5. Field test equipment.

ANALYSIS AND DISCUSSION OF TEST RESULTS

Laboratory Tests

Data obtained from the water and cement tests on concrete samples were analyzed to determine overall accuracy and the influence of aggregate type, aggregate moisture condition, concrete mix proportions, mix time, and sampling on test results. The percentage of recovery (measured values divided by actual values) was used as the basis of comparison, and the water tests were related to both the free and total water content of the mixes.

Table 2 gives the laboratory test results of the water and cement content of the concrete samples. Table 3 indicates that, for all batches, the average recovery was 97.8 percent for cement, 96.6 percent for free water, and 85.7 percent for total water. The associated standard deviations were 8.1 percent for cement, 4.4 percent for free water, and 3.7 percent for total water. The overall accuracies, including all the variables, were 8 and 4 percent respectively for the cement and water tests. Table 3 also indicates that the accuracies increased when each aggregate type was analyzed separately: The error in the cement tests decreased to about 6 percent, and the error in the water tests decreased to about 3.5 percent.

An analysis of variance was used to determine which parameters influenced the amounts of cement and water recovered. The parameters included in the analysis were aggregate type (coarse and fine quartz, coarse limestone and river sand, and coarse gravel and river sand), aggregate moisture condition (saturated plus some surface moisture and air dried), mix proportions [representing nominal 3,000, 4,500, and 6,000 psi (20.7, 31.0, and 41.4 MPa), mix time (5 and 45 min), and sampling sequence (sample obtained for water and cement content analysis before or after cylinder samples taken].

Results indicate that both the water and cement tests are sensitive at the 95 percent confidence level and are significantly influenced by the aggregate type. Average recovery values for cement ranged from a low of 93.5 percent for the quartz aggregate to a high of 104.8 percent for the limestone aggregate. Average water recovery values based on free water varied from 94.2 percent for quartz aggregate to 100.2 percent for gravel; conversely, water recovery based on total water varied from 83.5 percent for gravel to 89.1 percent for quartz.

The high cement-recovery value for the limestone aggregate concrete was attributed to the rock dust and limestone fines that passed through the nest of sieves above the washing machine. To confirm this, a cement test was conducted on a limestone aggregate sample representative of the limestone gradation and weight (420 g) used in the concrete specimens. The 420 g of limestone are equivalent to 12.5 g of cement or an error of 1.25 percent of cement. When this 1.25 percent is substracted from the cement test results, the mean cement recovery value for the limestone aggregate concrete is reduced to 96.59 percent.

In evaluating results of the water tests on the concrete samples, it was concluded that the test results are slightly more representative of free water than of total water; the recovery values based on free water are in all cases much closer to 100 percent.

Strength Prediction Based on Laboratory Results

Data obtained from the laboratory tests on concrete samples indicate that the chemical technique for determining water and cement content can be used directly to estimate the strength potential of a concrete mix. Figure 6 shows the 28-day cylinder strengths versus the water-cement ratios obtained in all batches tested. Figure 7 shows the 28-day cylinder strengths versus the actual water-cement ratios. [Actual water content is based on (a) free water available assuming the aggregates become saturated and (b) the quantity of mix water modified by the moisture content of the aggregate for each concrete batch.]

Table 2. Laboratory test results of water and cement content of concrete samples.

Batch No.	Batch Proportions (percent) Free Water	Total Water	Cement	Sample No.	Test Results (percent) Water	Cement	Recovery (percent) Free Water	Total Water	Cement
1	8.39	8.73	18.40	1	7.45	17.75	88.8	85.3	96.5
				2	8.15	17.80	97.1	93.4	96.7
2	8.80	9.18	14.24	1	7.80	13.30	88.6	85.0	93.4
				2	8.15	12.25	92.6	88.8	86.0
3	8.85	9.25	11.71	1	8.81	11.40	99.5	95.2	97.4
				2	8.45	12.65	95.5	91.4	108.0
4	8.64	9.02	14.24	1	8.15	13.25	93.2	84.7	93.3
				2	8.15	12.15	93.2	84.7	85.3
5	8.74	9.62	19.55	1	7.80	19.62	89.2	81.1	100.3
				2	8.15	20.05	93.2	84.7	102.6
6	8.54	9.47	15.14	1	7.80	16.87	91.3	82.4	111.4
				2	8.44	17.25	98.8	89.1	113.9
7	8.53	9.49	12.57	1	8.15	13.30	95.5	86.1	105.8
				2	8.15	13.38	95.5	86.1	106.4
8	8.42	9.35	15.14	1	7.47	15.58	88.7	79.9	102.9
				2	7.80	16.30	92.6	83.4	107.7
9	7.91	8.80	19.55	1	7.80	20.15	98.6	88.6	103.1
				2	7.80	19.15	98.6	88.6	98.0
10	8.02	8.95	15.14	1	7.65	16.50	95.4	85.5	109.0
				2	7.95	16.15	99.1	88.8	106.7
11	8.08	9.02	12.57	1	7.65	12.80	94.7	84.8	101.8
				2	8.15	14.40	100.9	90.3	114.6
12	8.02	8.95	15.14	1	7.45	15.00	92.9	83.1	99.1
				2	7.45	14.20	92.9	83.1	92.2
13	7.89	8.28	14.24	1	7.30	12.25	92.5	88.2	86.0
				2	7.30	13.80	92.5	88.2	96.9
14	7.88	8.27	14.24	1	7.45	12.25	94.5	90.1	86.0
				2	7.80	13.75	99.0	94.3	96.9
15	9.00	10.59	23.70	1	8.45	24.70	93.9	79.8	104.2
				2	8.45	21.25	93.9	79.8	89.7
16	9.12	10.80	16.84	1	8.82	16.45	96.7	81.7	97.9
				2	8.82	16.20	96.7	81.7	96.2
17	9.04	10.76	13.49	1	9.50	13.00	105.1	88.3	96.4
				2	9.16	10.25	101.3	85.1	76.0
18	8.97	10.65	16.79	1	9.16	15.90	102.1	86.0	94.7
				2	9.16	15.97	102.1	86.0	95.1
19	7.97	9.61	23.62	1	7.96	22.85	99.7	82.1	96.7
				2	8.15	22.45	102.1	84.1	95.0
20	7.98	9.69	16.79	1	7.96	15.55	99.7	82.1	92.6
				2	8.15	15.45	102.1	84.1	92.0
21	8.01	9.77	13.48	1	8.15	12.95	101.7	83.4	96.1
				2	8.15	13.18	101.7	83.4	97.8
22	7.96	9.68	16.79	1	8.15	16.15	102.4	84.2	96.2
				2	8.15	14.80	102.4	84.2	88.1

Table 3. Statistical analysis of recovery values of concrete samples.

Item	All Batches Mean	Standard Deviation	Batch 1-4, 13-14[a] Mean	Standard Deviation	Batch 5-12[b] Mean	Standard Deviation	Batch 15-22[c] Mean	Standard Deviation
Cement	97.79	8.06	93.53	6.77	104.77	5.96	94.03	6.03
Free water	96.56	4.40	94.17	3.68	94.87	3.62	100.23	3.25
Total water	85.70	3.74	89.11	3.82	85.35	3.09	83.50	2.27

[a]Maryland quartz aggregate. [b]Limestone-sand aggregate. [c]Gravel-sand aggregate.

Figure 6. Chemically determined water-cement ratio versus 28-day compressive strength for all batches.

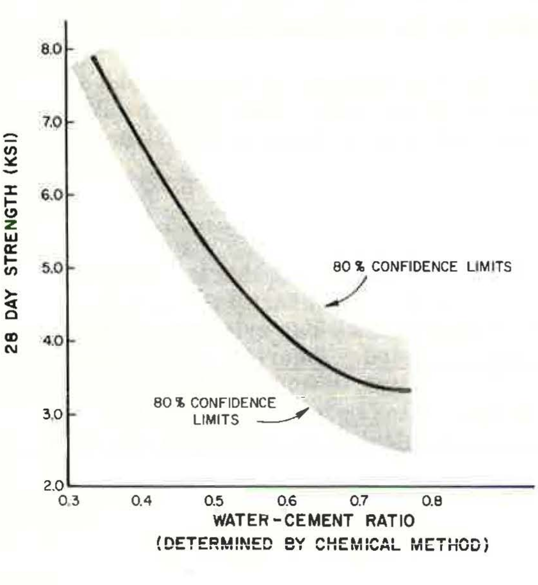


Figure 7. Actual water-cement ratio versus 28-day compressive strength.

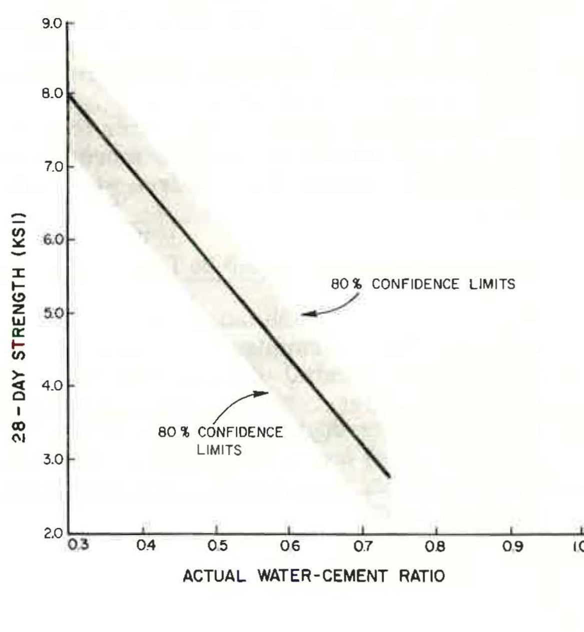


Table 4 gives the error associated with using the chemical technique to determine water and cement content as a measure of strength potential of a concrete. When all results are grouped together (Figure 6), the 80 percent confidence limits relating the chemically determined water-cement ratio to strength are ±780 psi (5.37 MPa). When results are grouped by aggregate type (Table 4), however, the confidence bands decrease to ±550, ±500, and ±350 psi (3.79, 3.44, and 2.41 MPa) for the quartz, gravel-sand, and limestone-sand aggregate combinations respectively. Similar trends and improvements were also noted when strengths were compared to actual water-cement ratios (Table 4).

Table 4 compares the confidence limits for predicting strength by the actual and the chemically determined water-cement ratios. This comparison indicates that when all three aggregate combinations are grouped together confidence bands for the actual and the chemically determined water-cement ratios are nearly equal [±780 psi (5.37 MPa) for the chemically determined versus ±720 psi (4.96 MPa) for the actual]. When the comparison is made individually by aggregate type, the spread of the confidence limits for the actual water-cement ratio values is less for two of the three aggregate combinations.

Another variable evaluated was strength within a batch. Within-batch strength variations are normally associated with discrepancies in mixer efficiency, fabricating, curing, and testing. The within-batch variation obtained for the complete concrete test series was 196 psi (1.35 MPa) for the 80 percent confidence limit.

All the above analyses indicate that the chemical procedure for determining water and cement content can be used to predict strength potential with an error no greater than if strength determination were based on the actual water-cement ratios of the mixes.

Field Tests

The field evaluations of the testing technique and the mobile unit have indicated that the unit can be transported with the automatic pipettes mounted in a ready-to-use configuration on the camper doors. Only one major equipment deficiency was noted during the field tests, the sensitivity of the flame photometer to external light. The use of a hood and side shields around the flame photometer decreased the sensitivity, but, even with the hood and shield, calibrating and holding calibration during the determination of an unknown cement solution were difficult. Present procedures permit the operations of the flame photometer inside the camper.

Table 5 gives the results obtained from the field tests and compares them to those for the mix designs. For the water content test, results indicate excellent agreement between the test results and the mix designs. The average recovery and associated standard deviation for the water test were 99.62 percent and 7.52 percent respectively. The results from the cement test were not quite so encouraging. For the cement test, the average recovery was 94.39 percent and the standard deviation was 26.6 percent. It is assumed that the flame photometer's sensitivity to external light was partially responsible for the higher deviations. In addition, the last 11 tests were conducted on a calcareous aggregate (both coarse and fine) concrete, requiring an aggregate blank test for removing the aggregate influence on the cement test results. This added another variable to influence cement test results.

The water-cement ratios obtained from the field tests were plotted against 28-day control cylinders. Figure 8 shows the field tests overlayed on the laboratory water-cement ratio versus 28-day cylinder strengths. The vast majority (12 out of 16) of the field test results fell near or within the 80 percent confidence limits of the laboratory test results. Of the four that fell outside, three were from the calcareous aggregate concrete. Even though the field data base is small and quite limited, the results indicate the potential of using the chemically determined water and cement content as a field test in evaluating concrete strength potential.

Table 4. Errors in strength predictions based on chemical technique, 80 percent confidence limits.

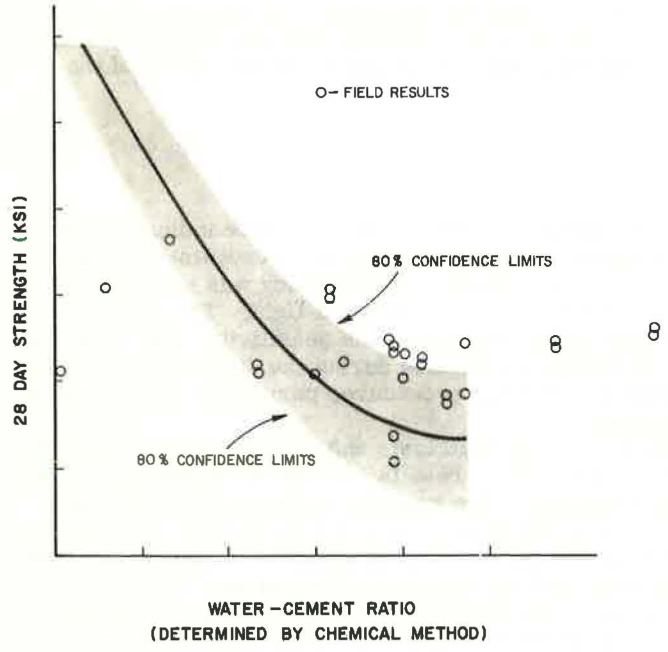
Sample Group	Error in Predicting Actual Water-Cement Ratio	Error in Test Data (psi)	Actual Strength Prediction (psi)
All	±0.078	±780	±720
Quartz	±0.060	±550	±175
Limestone-sand	±0.025	±350	±480
Gravel-sand	±0.046	±500	±335

Note: 1 psi = 6.9 kPa.

Table 5. Field test results of water and cement content of concrete samples.

Test No.	Mix Proportions (percent)		Test Results (percent)		Recovery (percent)	
	Water	Cement	Water	Cement	Water	Cement
1	9.85	19.8	10.5	15.5	106.5	78.4
2	9.85	19.8	9.5	15.2	96.0	76.8
3	9.86	19.8	9.2	15.4	93.3	77.7
4	7.21	19.2	7.1	16.9	99.0	87.7
5	7.21	19.2	6.83	19.5	94.7	101.3
6	6.92	12.4	7.48	10.3	108.0	83.0
7	7.60	11.8	7.15	10.3	94.0	87.2
8	7.60	11.8	7.80	10.2	102.5	86.5
9	7.60	11.8	8.15	11.0	107.0	93.2
10	7.60	11.8	7.80	12.8	102.5	108.5
11	7.60	11.8	8.15	11.9	107.0	100.7
12	7.60	11.8	8.15	15.4	107.0	130.5
13	7.60	11.8	7.15	8.1	94.0	68.6
14	7.60	11.8	8.15	11.4	107.0	96.6
15	7.60	11.8	7.15	7.2	94.0	61.0
16	7.60	11.8	6.2	20.4	81.5	94.39

Figure 8. Chemically determined water-cement ratio versus 28-day compressive strength compared with field test results.



CONCLUSIONS

The results of this study indicate the following:

1. A chemical procedure has been developed that can rapidly (≈15 min) determine the water and cement content of a concrete in the plastic state.
2. The chemical procedure for determining water and cement content can be used to predict the strength potential of the concrete. The reliability of predicting strength by this procedure is nearly equal to that of predicting strength based on actual mix proportions.
3. Aggregate type, such as limestone, gravel, or quartz, significantly influences the results obtained from the chemical tests. Although the chemical method is also sensitive to aggregate moisture condition, mix proportions, and length of mix time, the degree of sensitivity is for all practical purposes insignificant.
4. Even though the chemical method is sensitive to the type of aggregate used, satisfactory results were obtained for concrete made from both gravel and limestone coarse aggregate.
5. The one major limitation of the chemical method is that the cement content technique decreases in accuracy if the fine aggregate or sand has a high calcium content. This occurs when a manufactured sand (crushed limestone) is used for the fine aggregate.
6. Field tests have indicated that the system is fieldworthy and mobile.

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ACCELERATED SPLITTING-TENSION TEST FOR DETERMINING POTENTIAL 28-DAY SPLITTING-TENSILE STRENGTH AND MODULUS OF RUPTURE OF CONCRETE

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Recently, accelerated strength tests have been proposed for estimating the potential 28-day compressive strength of concrete. Attempts are also under way to find a means of replacing the 28-day compression test by one of the accelerated strength tests. This paper reports results of studies performed to determine if an accelerated test can be used to predict the potential 28-day modulus of rupture of concrete. In the modified boiling method adopted for this purpose, test cylinders are moist-cured for 18½ hours, boiled for 3½ hours, and tested in splitting tension 2 hours later. The companion 6 by 12-in. (15.2 by 30.5-cm) cylinders and 6 by 6 by 20-in. (15.2 by 15.2 by 50.8-cm) prisms are tested for splitting-tensile strength and modulus of rupture at 28 days. Correlation coefficients have been obtained for the results of accelerated splitting-tension and 28-day splitting-tension and modulus-of-rupture tests performed in both the laboratory and the field. From the limited test data available to date, it is concluded that the accelerated splitting-tension test can predict with sufficient accuracy the potential 28-day splitting-tensile strength and modulus of rupture of concrete.

•RECENTLY, accelerated strength tests have been used more frequently to determine potential 28-day compressive strengths. This was possible because of the pioneering researches of Akroyd (1), Malhotra (2, 3), and others (4). Accelerated strength tests are now being seriouly considered by the Canadian Standard Association to replace the 28-day standard acceptance test. However, little or no work has been reported to determine the feasibility of using a similar accelerated strength test to predict the potential 28-day modulus of rupture of concrete.

The present investigations were therefore undertaken to establish relationships between the accelerated splitting-tension tests and the 28-day splitting-tension modulus-of-rupture tests using standard test specimens and procedures. Preliminary experimental work was performed in a laboratory, and then the investigations were done in the field during the construction of street pavements in Monterrey City, Mexico.

LABORATORY INVESTIGATIONS

Materials Used

Normal portland cement (ASTM type 1) was used in the investigations. The physical properties of the cement are given in Table 1. The coarse aggregate was dolomitic limestone and had a maximum size of 1½ in. (3.8 cm). The fine aggregate was a natural sand. The bulk specific gravity and absorption of coarse and fine aggregate were 2.77 and 0.7 percent and 2.70 and 1.4 percent respectively.

Concrete Mixes

Six batches, covering wide water-cement ratios, were made in a 3-ft^3 (0.085-m^3) laboratory mixer. The mixes were designed to have a slump of 2 ± ½ in. (5.1 ± 1.3 cm). Nine test specimens consisting of six 6 by 12-in. (15.2 by 30.5-cm) cylinders and three 6 by 6 by 20-in. (15.2 by 15.2 by 50.8-cm) prisms were cast by hand-rodding from each batch by using standard ASTM procedures.

The standard 6 by 12-in. (15.2 by 30.5-cm) cylinders for the accelerated splitting-tension test were covered with tight-fitting steel plates; the cylinders and beams for 28-day tests were covered with a layer of impervious plastic.

After they were cast, all test molds were left in the laboratory at 73 F (23 C) and at a relative humidity of 65 percent for 18½ hours after which the cylinders and beams for 28-day testing were removed from the molds and transferred to a moist curing room.

The test cylinders for accelerated splitting-tension test were treated as follows. Three 6 by 12-in. (15.2 by 30.5-cm) cylinders while still in their molds were placed in a water tank maintained at 196 F ± 2 F (91 C ± 1 C). This temperature was selected to avoid excessive evaporation at boiling. After 3½ hours of curing at this temperature the cylinders were removed from the tank, demolded, and allowed to cool at room temperature for 1½ hours. The cylinders were then prepared for splitting-tensile strength and tested 30 min later. The total curing cycle was 24 hours.

The cylinders and beams for 28-day splitting-tension and modulus-of-rupture tests were tested in accordance with ASTM C 496-71 and C 78-72.

Analysis of Test Results

Plots of the results of the accelerated splitting-tension tests versus those of the 28-day splitting-tension and modulus-of-rupture tests are shown in Figures 1 and 2. Correlation coefficients and regression equations are also shown on these figures. Correlation coefficients for accelerated splitting-tensile strength and 28-day splitting-tensile and modulus-of-rupture values are greater than 0.98. This indicates highly significant correlations. This is even more remarkable, considering that water-cement ratios covering a wide strength range were used.

FIELD INVESTIGATIONS

After promising test results in the laboratory had been obtained, investigations were carried out to evaluate the reliability with which accelerated splitting-tensile tests could be used in the field to predict the later age splitting-tensile strengths and modulus-of-rupture values.

Table 1. Physical properties of type 1 portland cement used in laboratory investigations.

Physical Properties	Measurement
Fineness[a]	
Air permeability, cm^2/g	3300
Percent passing No. 325 mesh	92
Time of setting, hours and minutes	
Initial	2, 25
Final	3, 10
Soundness, percent	
Autoclave, expansion	0.14
Compressive strength of 2-in. cubes, psi	
3-day	2,560
7-day	3,510
28-day	4,960

Note: 1 in. = 2.5 cm. 1 psi = 6.9 kPa.

[a]Property supplied by the cement manufacturer.

Figure 1. Accelerated-cured splitting-tensile strength versus 28-day standard-cured splitting-tensile strength for laboratory tests.

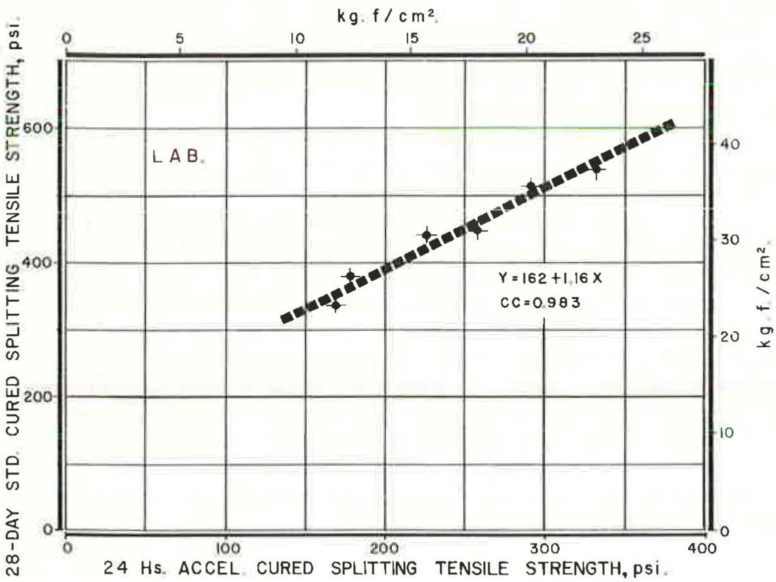
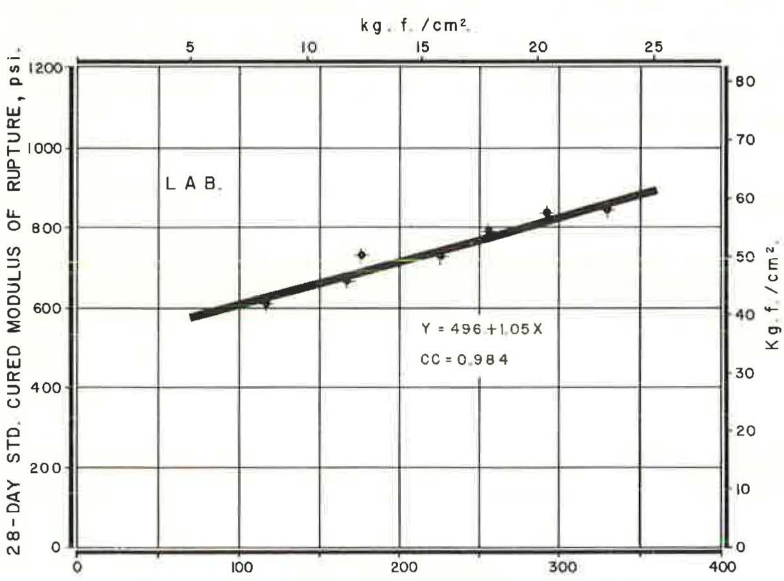


Figure 2. Accelerated-cured splitting-tensile strength versus 28-day standard-cured modulus of rupture for laboratory tests.



Materials Used

Portland blast furnace slag cement (ASTM C 595-73) was used for investigations in the field. The chemical analysis and physical properties of the cement are given in Table 2. The coarse aggregate was crushed blast furnace slag and had a maximum size of 1½ in. (3.8 cm). Its chemical analysis is also given in Table 2. The fine aggregate was crushed limestone sand. The bulk specific gravity and absorption of coarse and fine aggregates were 2.40 and 4.5 percent and 2.63 and 1.5 percent respectively. A water-reducing admixture was also incorporated in the mixes.

Mixing, Sampling, and Curing Test Specimens

The concrete was batched in a dry mix plant and transit mixed in 6-yd^3 (4.6-m^3) mixers. The slump and air content of the mix were maintained at 2 ± ½ in. (5.0 ± 1.3 cm) and 4 ± 1 percent respectively. All sampling was carried out in accordance with ASTM C 172-71.

Six specimens were cast from each batch sampled. They consisted of four 6 by 12-in. (15.2 by 30.5-cm) cylinders for the splitting-tension test and two 6 by 6 by 20-in. (15.2 by 15.2 by 50.8-cm) prisms for modulus-of-rupture test. A total of 30 batch samples were obtained.

The cylinder and beam specimens were cast in steel molds in accordance with ASTM C 31-69, and compaction was achieved by hand-rodding. The cylinder molds for accelerated curing were covered with tight-fitting steel plates, and the remaining molds were covered with wet burlap.

After they were cast, all molded specimens were left at the site for 16 hours and then were transported to the university laboratory. The molded specimens for accelerated splitting-tensile strengths had at least 18½ hours of curing before they were placed in the hot-water tank. The procedures for the modified curing cycle used for accelerated strength tests and the 28-day tests were the same as those described for laboratory investigations.

Analysis of Test Results

Plots of results of accelerated splitting-tension test versus those of 28-day splitting-tension and modulus-of-rupture tests and correlation coefficients and regression equations are shown in Figures 3 and 4. The correlations are generally less than 0.75. These correlation coefficients are not so high as those obtained in the laboratory; this is to be expected. Nevertheless, with more stringent control of the concrete mix proportions at the ready-mix concrete plants, the degree of correlations should improve.

Within-Batch and Between-Batch Variations

The within-batch and between-batch variations for the test results are given in Table 3. These values seem to be reasonably satisfactory for specimens cast in the field.

CONCLUDING REMARKS

From the limited investigations discussed it can be concluded that the accelerated splitting-tension test offers possibilities of predicting 28-day splitting-tensile strengths and modulus-of-rupture values. The predicted values fall within ±15 percent limits.

In the investigations reported, only the 24-hour cycle has been tested for accelerated curing. Further research is needed to optimize the curing cycle so that the accuracy with which later age strengths can be predicted can be improved.

Table 2. Physical properties and chemical analysis of portland blast furnace slag cement and chemical analysis of blast furnace slag aggregate used in field investigations.

Item	Measurement
Physical properties of cement	
Fineness[a]	
Air permeability, cm^2/g	4200
Percent passing No. 325 mesh	97
Time of setting, hours and minutes	
Initial	2, 45
Final	3, 00
Soundness, percent	
Autoclave, expansion	0.02
Compressive strength of 2-in. cubes, psi	
3-day	2,770
7-day	3,840
28-day	6,120
Chemical analysis of cement, percent	
CaO (total)	55.50
SiO_2	25.50
Al_2O_3	7.70
Fe_2O_3	2.50
SO_3	1.83
Lost by ignition	1.20
Alkali (total)	0.44
No soluble remnant	0.20
Chemical analysis of aggregate, percent	
CaO	42.00
SiO	33.92
Al_2O_3	13.24
MgO	9.52
S	2.12
TiO_2	0.84
Fe_2O_3	0.72
P and Mn	0.14

Note: 1 in. = 2.5 cm. 1 psi = 6.9 kPa.

[a]Property supplied by cement manufacturers.

Figure 3. Accelerated-cured splitting-tensile strength versus 28-day standard-cured splitting-tensile strength for field tests.

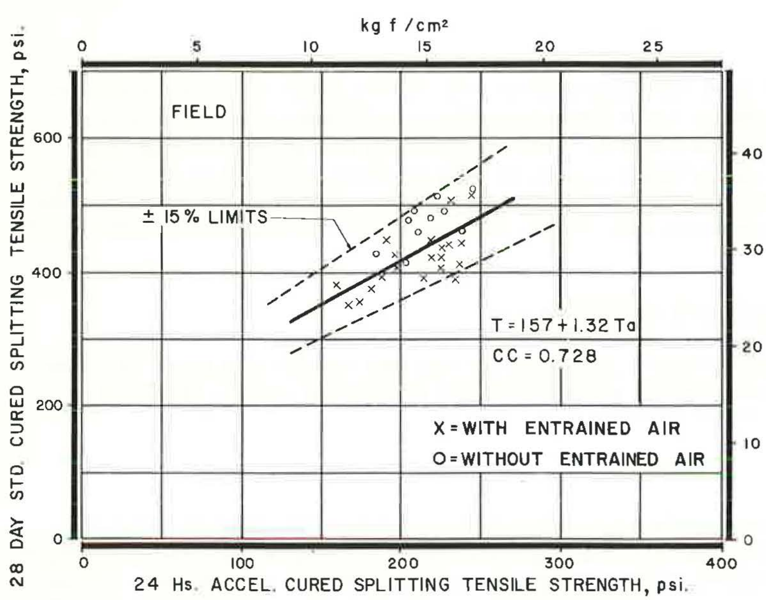


Figure 4. Accelerated-cured splitting-tensile strength versus 28-day standard-cured modulus of rupture for field tests.

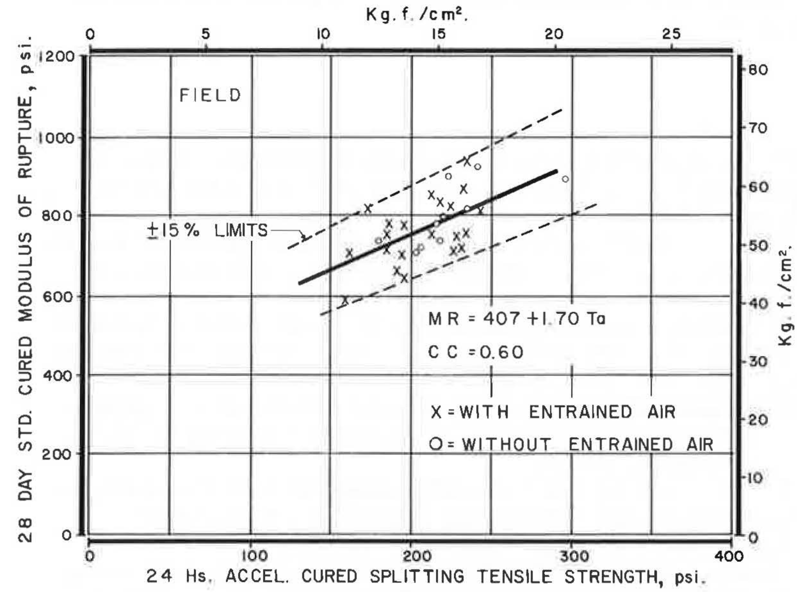


Table 3. Evaluation of within-batch and between-batch variations for field investigations.

Test	No. of Tests	Mean (psi)	Avg Within-Batch Variation: Range (psi)	Avg Within-Batch Variation: V_1 (percent)	Avg Between-Batch Variation: Standard Deviation (psi)	Avg Between-Batch Variation: Coefficient of Variation (percent)
28-day						
Modulus of rupture	52	768	54.0	6.16	23.3	16.06
Splitting tension	52	492	33.8	6.01	55.9	11.84
Accelerated						
Splitting tension	30	207	10.9	4.69	30.0	14.43

Note: 1 psi = 6.9 kPa.

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