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An Evaluation of the Maturity Method (ASTM C 1074) for Use in Mass Concrete

by *Toy S. Poole, Patrick J. Harrington*

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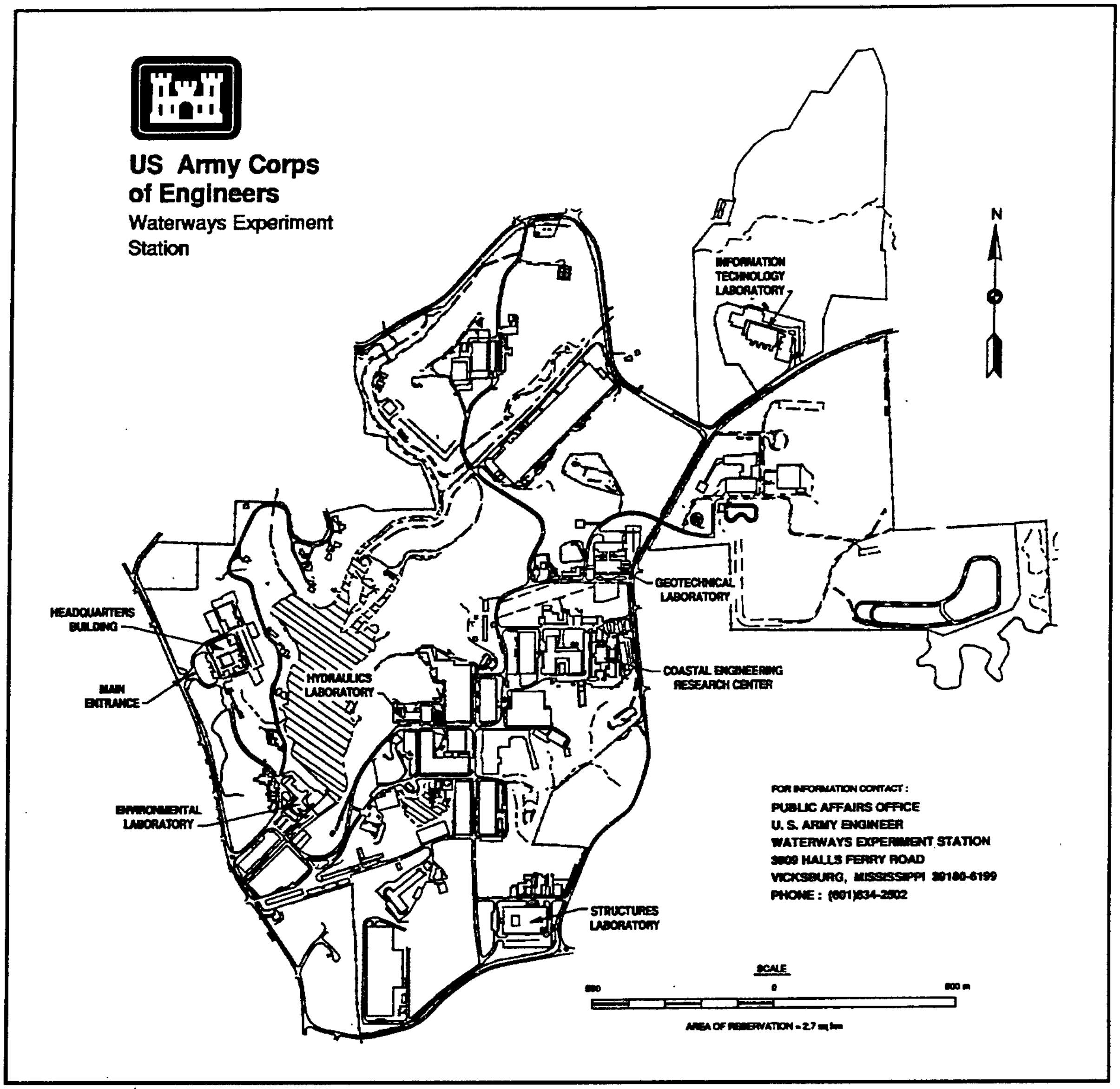

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Contents

Preface

The investigation described in this report was conducted by the Concrete and Materials Division (CMD), Structures Laboratory (SL), U.S. Army Engineer Waterways Experiment Station (WES). The work was sponsored by Headquarters, U.S. Army Corps of Engineers, as part of Civil Works Investigation Studies Work Unit 31138, "New Technologies for Testing and Evaluating Concrete."

The study was conducted under the general supervision of Messrs. Bryant Mather, Director, SL, and John Q. Ehrgott, Assistant Director, SL. Dr. Paul F. Mlakar was Chief, CMD, during this work. Direct supervision was provided by Mr. Edward F. O'Neil, Acting Chief, Engineering Mechanics Branch (EMB), CMD. MAJ Patrick J. Harrington was the principal investigator during the planning and data-collection phases of the work. Messrs. Anthony A. Bombich and Richard Haskins, EMB, designed and supervised the placement of instrumentation and automatic data collection. MAJ Harrington performed most of the maturity calculations, and Dr. Toy S. Poole analyzed the data and wrote the report.

At the time of the publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

1 Introduction

Background

The objective of this work was to investigate the applicability of American Society for Testing and Materials (ASTM) C 1074,[1] "Standard Practice for Estimating Concrete Strength by the Maturity Method" (ASTM 1992h), to the estimation of in-place strength of mass concrete containing large amounts of fly ash. To accomplish this, the U.S. Army Engineer Waterways Experiment Station (WES) and U.S. Army Engineer District, Vicksburg, cooperated in an investigation involving concrete construction at Red River Waterway Lock and Dam No. 4 (L&D 4). This investigation included laboratory determination of maturity parameters of the concrete mixture used at L&D 4, then compared strengths predicted from maturity measurements with strengths measured on cores taken from two monoliths that had been instrumented for temperature measurements.

The Maturity Method

Even though strength development in lab concrete is usually determined with time as the single independent variable; i.e., temperature is held constant, it is well known that strength development of in-place concrete is dependent on both time and temperature. The summation of the time-temperature history of concrete determines its maturity. Because time-temperature history of in-place concrete can differ considerably from the time-temperature history of similar laboratory-cured specimens, strength development of the laboratory-cured specimens is often a poor indicator of strength development of the in-place concrete. Field curing of specimens is an established practice (e.g., ASTM C 31 (ASTM 1992a), ASTM C 873 (ASTM 1992g)) that seeks to reduce this discrepancy and these methods provide a reasonable solution to this problem when the dimensions of the concrete structure are such that ambient temperature is the principal factor controlling the temperature of both the in-place concrete and the

[1] This work was done prior to 15 Sep 93 and used C 1074-87, which was replaced on 15 Sep 93 by C 1074-93, which was revised in Aug 95 to correct Equation 2.

test specimens. In the case of mass concrete, the ambient temperature is only one of several factors controlling concrete temperature, with size of placement, rate of heat evolution, and location in the placement being others. Therefore, the time-temperature history of mass concrete, and consequently the strength development, may vary considerably from that of field-cured specimens and may vary throughout the structure. This complexity makes use of laboratory-cured or field-cured specimens essentially useless in predicting in-place strength. However, if temperature history can be determined and a reasonable functional relationship between strength and this time-temperature history can be developed, then strength development can be estimated from this temperature history. There has been and continues to be considerable research on the mathematical relationships appropriate for this purpose (Kjellsen and Detwiler 1993; Carino and Tank 1992; Chengju 1989; Carino, Knab, and Clifton 1992; Carino 1984). Malhotra and Carino (1991) provides a good description of much of this. ASTM C 1074 (ASTM 1992h) is a practice that uses two of these approaches.

Literature on the application of maturity concepts to prediction of strength of concrete date to the late 1940's (Chengju 1989; Oluokun, Burdette, and Deatherage 1990). In these early efforts, maturity was represented as the simple sum of the number of degree-days above 0 °C, the minimum temperature at which cement was believed to hydrate. The method was calibrated by measuring the strength of laboratory concrete specimens whose time-temperature history was known. This approach, with some modifications, was useful as long as concrete temperatures did not exceed about 30 °C. Beginning in the 1960's, efforts were made to relate the effect of time-temperature history to more sophisticated thermodynamic principles. ASTM C 1074 (ASTM 1992h) includes procedures representing both of these approaches.

Representing the older, simpler approach, ASTM C 1074 uses the following maturity function:

$$M(t) = \sum (T_a - T_o)\Delta t \tag{1}$$

where

$M(t)$ = time-temperature factor at time t
T_a = average concrete temperature over a time interval (Δt)
T_o = datum temperature

Datum temperature is a constant that is determined for each concrete and represents the minimum temperature at which cement hydrates. In the absence of an empirically determined value, the method recommends using 0 °C. By use of a set of calibration specimens, the time-temperature factor is related to strength development. From this calibration curve, strengths can be predicted for a given time-temperature factor of in-place concrete.

In the other approach described in ASTM C 1074 (ASTM 1992h), an equivalent age (t_e) is calculated:

$$t_e = \sum \Delta t \cdot e^{-Q\left[\frac{1}{T_a} - \frac{1}{T_o}\right]} \tag{2}$$

This work was done prior to 15 Sep 1993 and used ASTM C 1074-87, which was replaced on 15 Sep 1993 by ASTM C 1074-93, which was revised in Aug 1995 to correct Equation 2

where

Δt = time interval between temperature readings
T_a = average temperature over the interval
T_o = reference temperature
Q = constant specific to each concrete

The reference temperature can be any value, but the same value is used throughout all calculations. A calibration curve is prepared from strengths of specimens whose time-temperature history is known. Then from time-temperature data taken from in-place concrete, strength can be estimated from this calibration curve.

The principle underlying this procedure is that the dependence of rate of strength gain on temperature follows the Arrhenius equation:

$$k = Ae^{-\frac{E_a}{RT}} \tag{3}$$

where

k = rate constant of the strength versus time relationship
E_a = activation energy
R = gas constant
T = temperature (°K)
A = frequency factor and does not enter into the maturity calculations

ASTM C 1074 simplifies this equation by defining a parameter, Q, which is equal to E_a/R. The method gives procedures for determining Q empirically, or, in the absence of such determinations, default values are recommended.

In this work, calculations were done with both approaches, on laboratory prepared and cured specimens, and on two concrete monoliths that had been instrumented for temperature measurements. Cores were then taken from locations near the temperature sensors to determine how well predicted strengths conformed to actual strength in these monoliths.

2 Materials and Methods

Materials and Concrete Mixture Properties

Concrete was made from Type II portland cement that had been specified to meet the heat-of-hydration requirement of ASTM C 150 (ASTM 1992c) (290 kj/kg at 7 days). This cement has lower C_3A and C_3S contents than commonly available Type II cements and is ground to a coarser particle size. These properties are necessary to meet the heat-of-hydration requirement. Properties of a sample of this cement used in the laboratory-prepared calibration specimens are described in Table 1. Fly ash met requirements of a Class C pozzolan (ASTM C 618 (ASTM 1992f)). Properties of a sample used in the laboratory determinations are also described in Table 1. The concrete mixture used to make the laboratory specimens is described in Table 2. This mixture was the same as that used in the monolith construction.

Table 1
Chemical and Physical Properties of Portland Cement and Fly Ash

Chemical Analysis	Portland Cement	Fly Ash	Physical Property	Portland Cement	Fly Ash
SiO_2, %	21.2	35.1	Fineness, AP, m^2/kg	290	--
Al_2O_3, %	5.0	19.0	Initial Set, min	190	--
Fe_2O_3, %	6.5	8.5	Final Set, min	340	--
CaO, %	61.6	--	Air, %	9	--
MgO, %	0.9	4.9	3-day Str., MPa	11.8	--
SO_3, %	2.6	2.3	7-day Str., MPa	16.9	--
LOI, %	0.8	0.3	Heat of Hydr. kj/kg, 7 days	259	--
Insol. Res., %	0.11	--	Fineness, %	--	18
Na_2O, %	0.10	--	Density, Mg/m^3	--	2.55
K_2O, %	0.46	--	Str. Index, %, 7 day	--	85
C_3A, %	3	--	Water Req, %	--	94
C_3S, %	37	--	Soundness, %	-0.01	0.00
C_2S, %	33	--			
C_4AF, %	20	--			

Calibration Specimens

Twenty-four 152- × 305-mm cylinders were cast for compressive strength determinations and two similar cylinders were cast with embedded temperature sensors in their center. The specimens were cured in a moist room meeting the requirements of ASTM C 511 (ASTM 1992e). Compressive strengths were determined at 1, 3, 7, 14, 27, 56, and 90 days according to ASTM C 39 (ASTM 1992b).

Table 2
Mixture Proportions for Laboratory Concrete, Mixture No. B4

Component	SSD Mass or vol/m³
Portland Cement	162 kg
Fly Ash	71 kg
Fine Aggregate	711 kg
Coarse Aggregate	637 kg
Coarse Aggregate	642 kg
Air-Entraining Admixture	104 mL
Water	125 kg

Temperature and Strength Measurements of In-Place Concrete

Two placements were instrumented with thermocouples to measure time-temperature history. These were in monoliths 6/7 (ML6/7), placed on 16 March 93, and monoliths 14 and 15 (ML14/15), placed on 16 August 93. The positioning of thermocouples is illustrated in Figures 1a and 1b. Cores were taken near the thermocouples at 4, 7, 14, 28, and 52 days after placing for ML6/7 and at 4, 7, 14, 28, and 44 days after placing for ML14/15. Cores were cut into 102- × 203-mm cylindrical specimens and compressive strength determined according to ASTM C 39 (ASTM 1992b). The mean strength of three cores was taken as a value for each age.

Eight thermocouples were placed in each monolith, as illustrated in Figure 1. A Quadrel Calorimeter data logger was used to accumulate time-temperature data from four of them (no. 2, 4, 6, and 8). Temperature of the concrete was taken as the average of the four measurements. Data were logged every 0.25 hr from initial concrete placement through 14 days, then every 0.5 hr through the end of the test. Data from ML6/7 were collected through 52 days and data from ML14/15 through 44 days. Temperature data for laboratory-cured specimens used for calibration were collected every 0.25 hr through 12 days, then every 0.5 hr through 36 days, then every 100 hours through 91 days.

3 Results and Discussion

Empirical Determination Q and Datum Temperature

Q and datum temperature (DT) were determined empirically for both a mortar containing fly ash and a mortar containing no fly ash, using strength development of mortar cubes, as directed in ASTM C 1074 (ASTM 1992h). The no-fly ash condition was not pertinent to this work but was included so that values of Q determined could be compared to literature values as a plausibility check. Curing temperatures were 10, 23, and 38 °C. Fly ash mortar batches contained 325 g of cement, 146 g of fly ash, 1,460 g of project sand, and a water to cement plus fly ash ratio (w/c+fa) of 0.54 by mass. Portland cement mortar batches contained 500 g of cement, 1,550 g of sand, and a w/c of 0.54. Materials were preconditioned at their respective temperatures prior to mixing and kept at those temperatures until time of strength testing. Time of setting (ASTM C 191 (ASTM 1992d)) was determined at each temperature and was used as the basis for determining ages at which strength would be determined (para A1.1 of C 1074 (ASTM 1992h)). Three mortar cubes were broken at each age. Mean strengths are summarized in Table 3.

Table 3
Mortar-Strength Data Used in Calculating Q
Age is in Days, Strength (str) is in MPa

0% fly ash 10 °C TOS=9 hr		0% fly ash 23 °C TOS=6 hr		0% fly ash 38 °C TOS=4 hr		35% fly ash 10 °C TOS=15 hr		35% fly ash 23 °C TOS=8.5 hr		35 % fly ash 38 °C TOS=5 hr	
age	str	age	str	age	str	age	str	age	str	age	str
0.75	0.33	0.50	0.43	0.33	0.90	1.25	0.23	0.71	0.60	0.42	0.94
1.7	3.5	1	3.4	0.83	7.9	2.50	2.4	1.6	5.9	0.83	6.0
3	6.0	2	8.6	1.3	12.1	5	7.1	2.8	7.4	1.9	9.8
6	10.7	4	13.4	2.8	17.0	10	10.7	5.6	11.4	3.3	15.3
12	15.4	8	17.2	5.3	18.8	20	14.4	11	15.1	6.9	20.4
24	19.6	16	22.4	11	25.8	40	16.5	23	22.4	13	34.0

Table 4 summarizes the coefficients obtained from the linear regression of reciprocal time on reciprocal strength, along with K values, as described in paragraph A1.1.8.1 of ASTM C 1074 (ASTM 1992h). Intercepts of these relationships were negative, which is not realistic since this constant represents the reciprocal of strength at infinite time for each temperature. Also, negative values of the intercept would result in negative rate constants. The intercept of a linear regression calculations is strongly affected by the value of strength measured at very early ages. If the relationship between reciprocal time and reciprocal strength is slightly nonlinear due to early-age effects, then the error in the intercept can be quite large. As is illustrated in Figure 2, this appears to be strongly the case. The problem lies in the saturation function ASTM C 1074 uses to fit strength-time data. This function poorly fits the data generated in this work. Either some refinements in methodology or use of a different function are necessary. In an attempt to remove this nonlinearity effect, the calculations were also done after omitting the earliest-age data point from each curve. This improves the linearity of the curve and gives positive values for the rate constants. This data-rejection procedure is not part of the standard procedure.

Table 4
Summary of Regressions of 1/time ($days^{-1}$) vs. 1/strength (MPa^{-1})

	0% fly ash 10 °C	0% fly ash 23 °C	0% fly ash 38 °C	35% fly ash 10 °C	35% fly ash 10 °C	35% fly ash 10 °C
Slope, days/MPa	2.3126	1.1435	0.3650	5.2650	1.1832	0.4242
Intercept, MPa^{-1}	-0.3595	-0.2623	-0.0987	-0.5343	-0.1652	-0.0827
r, corr. coeff.	0.9421	0.9286	0.9518	0.9209	0.9436	0.9365
K, $days^{-1}$	-0.1554	-0.2294	-0.2704	-0.1015	-0.1396	-0.1950
Earliest Data Point Omitted						
Slope, days/MPa	0.4391	0.2622	0.0734	0.9683	0.2222	0.1179
Intercept, MPa^{-1}	0.0253	0.0156	0.0337	0.0078	0.0445	0.0293
r, corr. coeff.	0.9984	0.9824	0.9886	0.9695	0.9857	0.9913
K, $days^{-1}$	0.0576	0.0596	0.4591	0.0081	0.2003	0.2489

Estimates of Q and DT are summarized in Table 5. Standard error in Q is the standard error of the slope of the regression of ln K on 1/T, as described in paragraph A1.3 of ASTM C 1074. Standard error in DT is the standard error of the intercept of the regression of K on T, as described in paragraph A1.2 of ASTM C 1074.

Estimates of DT reported in the literature vary from 0 °C to -10 °C (Chengju 1989). Estimates of Q for portland cement containing no fly ash vary from about 4,000 K to 6,000 K (Chengju 1989; Malhotra and Carino 1991; Kjellsen and Detwiler 1993; Carino and Tank 1992). Carino and Tank (1992) report a value of Q of 3,750 K for portland cement containing 20-percent fly ash and a value of Q of 6,700 K for a mixture of 50-percent portland cement and 50-percent slag. Values determined as part of this work for the fly ash mixture differ largely from these values. Values of Q for the no-fly ash mixture are plausible.

Table 5
Summary of Q Values and Datum Temperatures (DT) used in Maturity Calculations

	Q (K)	Standard Error	DT (°C)	Standard Error
Recommended by ASTM C 1074, Type I cement	5,000	--	0	--
Measured, 0% fly ash	6,764	3,613	-0.048	0.087
Measured, 35% fly ash	10,693	5,774	-0.155	0.200

The uncertainty in these estimates is very high, as indicated by the large standard errors, therefore, it cannot be determined whether these differences are due to random error or to some source of bias in the determinations.

Errors in determinations of Q derive from errors in determining the rate constants at each temperature, K, and have a large random error component due to the fact that Q is calculated from a linear regression of only three values of K on reciprocal time. The uncertainty in such a calculation will be large unless the uncertainty in each of the three points is very low, which it is not.

An estimate of Q based on more data points would be more robust to errors in the estimates of each point, but the effort involved in generating enough data to reduce the uncertainty to reasonable levels would seriously detract from the method. As will be discussed below, there is possibly a way to apply this method that minimizes the need to have a very accurate estimate of Q.

Time-Temperature History

The temperature history of the laboratory-cured calibration mixture is illustrated in Figure 3. Temperatures varied over a relatively narrow range (23.2 to 25.4 °C).

The temperature history of ML6/7, which was placed in March, 1993, is illustrated in Figure 4. Initial temperature was 17 °C, increasing to a maximum of about 32 °C after 3 days. By 10 days, temperatures had approximately returned to the starting temperature. Temperatures from 10 to 52 days, when the testing was stopped, ranged from about 15 to 23 °C.

The temperature history of ML14/15, which was placed in August, 1993, is illustrated in Figure 5. The entire temperature history of this monolith was substantially warmer than ML6/7. Initial temperatures were about 30 °C, rising to about 49 °C after 3 days. Temperatures decreased more slowly than in L- 6/7, cooling to the initial temperature after about 30 days. Temperatures between 30 and 44 days, when testing was stopped, ranged from 25 to 32 °C.

Maturity Calculations

Maturity calculations and strengths of specimens used for making the calibration curve are summarized in Table 6. Calculations were done for a Q value of 10,693 k, as determined empirically, and for a Q value of 5,000 k. The latter is the value of Q recommended by ASTM C 1074 (ASTM 1992h), if no empirically determined value is known. A reference temperature of 20 °C was used. The slight elevation of the curing temperature over the reference temperature caused the equivalent ages to be somewhat accelerated relative to real time.

Time-temperature calculations were done with a value of DT of -0.115 °C, determined empirically, and a value of DT of 0 °C. The latter value is recommended by ASTM C 1074, if no empirical value has been determined.

Table 6
Maturity Values and Strength Data for Calibrations Specimens

Age, real time, days	Equivalent Age, days		Time-Temperature Factor, degree-days		Observed Strength, MPa
	Q=10,693	Q=5,000	DT[1]=-0.155	DT=0	
1	1.8	1.3	25.0	25.0	1.7
3	5.0	3.8	72.7	72.5	4.8
7	11.2	8.7	167.2	166.8	6.9
14	22.2	17.4	333.3	332.6	11.2
27	42.6	33.4	641.8	640.5	14.8
56	85.8	68.4	1,317.3	1,314.6	22.8
90	136.3	109.3	2,165.9	2,161.4	27.2

[1]DT = datum temperature

Maturity calculations for concrete in ML6/7 and ML14/15 are summarized in Tables 7 and 8, respectively. As with the calibration mixture, calculations are shown with empirically determined values of Q and DT, as well as with default values of the parameters.

Predicted vs Observed Strength

Data from Table 6 were used to construct calibration curves between either equivalent age or time-temperature factor and strength. Conveniently, it was found that the relationships between strength and the square root of equivalent age, and strength and the square root of time-temperature factor, were linear.

Table 7
Maturity Calculations and Strength Data for L- 6/7

Age, real time, days	Equivalent Age, days		Time-Temperature Factor, degree-days		Observed Strength MPa
	Q=10,693	Q=5,000	DT=-0.155	DT=0	
4	12.5	6.7	114.6	114.4	8.9
7	18.7	10.8	191.2	190.9	11.4
14	24.6	17.3	321.4	320.7	18.0
28	35.6	29.8	574.9	573.6	25.4
52	55.7	51.7	1,016.7	1,014.2	36.1

Table 8
Maturity Calculations and Strength Data for ML14/15

Age, real time, days	Equivalent Age, days		Time-Temperature Factor, degree-days		Observed Strength MPa
	Q=10,693	Q=5,000	DT=-0.155	DT=0	
4	81.2	14.2	183.3	183.1	8.5
7	131.3	27.3	316.5	316.1	18.0
14	184.7	45.2	575.5	574.9	23.7
28	233.2	73.7	1,032.4	1,033.0	29.0
44	292.8	99.2	1,485.9	1,483.8	31.6

This allowed strengths to be predicted from a given value of maturity from a simple equation. These calibration curves and linear regression equations are shown in Figures 6 and 7. These equations can be used to calculate predicted strengths from equivalent-age or time-temperature values in Tables 7 and 8. Predicted versus observed strengths are shown in Figures 8 through 11. In order to evaluate the effects of variations in estimates of Q, calculations were made with several Q values other than 5,000 k and 10,693 k, as indicated in these figures.

Review of quality control data on cylinders cast at the project suggested that the concrete mixture used in ML6/7 was not the same as used to fabricate the calibration specimens, thus probably causing an error in the predicted strengths for this placement. These QC data are presented in Figure 12. Results were recalculated using the quality control specimens as calibration specimens. The assumption was made that they experienced approximately the same temperature history as shown in Figure 3, since the same storage area was used. Revised predictions are presented in Figures 13 and 14.

Using the time-temperature approach, predicted strengths were consistently lower than measured strengths. The errors in prediction of strength in ML6/7 were large, ranging from -34 percent at 4 days to -50 percent after 52 days, as illustrated in Figure 9. When QC data were used to calibrate the method, these early-age errors became small, but later ages were still seriously underestimated (Figure 13). The errors in predicting strength in ML14/15 were relatively small at early ages, e.g., -10 percent at 4 days, but increased to about -30 percent at later ages, as illustrated in Figure 11. These early-age errors are similar to those found by Parsons and Naik (1985) using this method. As mentioned earlier, this method is not expected to give good results for the amount of temperature change that occurred in these concrete placements. Our data appear to confirm this.

Using the equivalent-age approach, two things are immediately apparent on examination of the predicted versus measured strength curves. One is that predictions were generally better at early ages than at later ages. The other is that the value of Q used in calculations was not very important in ML6/7 calculations while it was very important in ML14/15 calculations.

The errors in predicting strengths were worse for ML6/7. Strength were underestimated by 40 to 50 percent (Figure 8). This was improved substantially by using production QC data for calibration purposes (Figure 14), particularly at early ages. Errors in predicting strength in ML14/15 were small at 4 days when commonly accepted values of Q (5,000 and 6,000 k) were used, but errors were still large at later ages (Figure 10). Predictions made using the empirically-determined Q for ML14/15 were very high at all ages (Figure 10).

The divergence of the predicted strength from the measured strength could be caused by at least two things. One is that there may have been some variation in concrete mixture proportions and materials properties between the concrete used in making the calibration specimens and the concrete placed in the structure. Available data on QC specimens (Figure 12) suggests this might have been a contributing factor. The other potential source of error is that Q is not truly a constant but actually a variable whose value is dependent on time-temperature history of the concrete. The calibration specimens were cured at a relatively uniform temperature that did not differ very much from the reference temperature. The in-place concrete experienced a sizable temperature increase for the first few days after placement. It is well known that early temperature history of hydrating portland cement has effects on properties at later ages that go beyond a simple acceleration or deceleration of hydration during that early time. Therefore, the value of Q determined for concrete near 20 °C may be inappropriate for concrete that experienced such a large early-age rise.

Even assuming Q to be a true constant, the value of Q used in calculations had a large effect on predicted strengths in ML14/15 but had insignificant effects on predicted strengths in ML6/7. This is due to the differences in time-temperature history between these monoliths. The temperature of both monoliths showed a substantial early-age increase, but in ML6/7, the initial temperature was below the reference temperature (20 °C) and the peak early-age temperature was only about 12 deg above this. For most of the time data were collected, the monolith temperature and the reference temperature were very close together. The result of this relative conformity between these temperatures was that the part of the exponent of the maturity equation that is calculated from the difference between the reciprocal of the reference temperature and the reciprocal of the monolith temperature was usually a relatively small number and the value of Q used was then relatively unimportant. In the case of ML14/15, the initial temperature was about 10 deg above the reference temperature, the peak temperatures were close to 30 deg above the reference temperature, and the average temperature during the latter part of data collection was still about 10 deg above the reference value. Therefore, the temperature-difference part of the maturity equation was relatively larger and the value of Q used had a larger impact on calculations.

Given the large uncertainties in measuring Q, that Q probably varies with time and temperature, and the importance of the value of Q used when large temperature increases occur, a considerable improvement in the practical application of the technique would probably be realized if the specimens used to generate the calibration curves relating maturity to strength experienced a time-temperature history similar to that expected in the in-place concrete. For example, in this project, calibration specimens given a temperature increase of about 20 °C, gradually applied and removed over a 10-day interval would probably work well. Then, since both in-place and calibration concretes experience similar time-temperature histories, a reference temperature could be chosen that would minimize the size of the temperature-difference part of the maturity equation for both concretes, thus minimizing the importance of the value of Q used. This approximate temperature matching would hopefully also avoid the effect of the apparent time and temperature dependency of Q.

4 Conclusions and Recomendations

The time-temperature procedure in ASTM C 1074 (ASTM 1992h) is probably not suited for use in estimating in-place strength in concrete where large temperature rises occur. However, early-age strength might be estimated well, particularly if some effort were made to approximately match the early temperature history of the calibration specimens with that expected in the in-place concrete.

Large uncertainties are associated with empirical determination of Q, the activation energy divided by the gas constant. An accurate estimate would be difficult to obtain. Results of this work suggest that accurate values of Q are necessary if the concrete temperatures differ by more than about 15 °C from the reference temperature, either in the calibration specimens or in the in-place concrete, or both. Approximate temperature matching between calibration specimens and temperatures expected of in-place concrete would eliminate the need for accurate values of Q.

Using approximate temperature matching of calibration specimens to in-place concrete, early-age strength would be expected to be predicted reasonably well. It is not clear that this procedure change would also fix the problem with the later-age predictions. Additional research is needed to verify that these recommendations would be effective and to establish levels of precision and bias expected in the method.

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d. Designation C 191-92. "Standard test method for time of setting of hydraulic cement by vicat needles."

e. Designation C 511-85. "Standard specification for moist cabinets, moist rooms, and water storage tanks used in the testing of hydraulic cements and concretes."

f. Designation C 618-92a. "Standard specification for fly ash and raw or calcined natural pozzolan for use as a mineral admixture in portland cement concrete."

g. Designation C 873-85. "Standard test method for compressive strength of concrete cylinders cast in place in cylindrical molds."

h. Designation C 1074-87. "Standard practice for estimating concrete strength by the maturity method."

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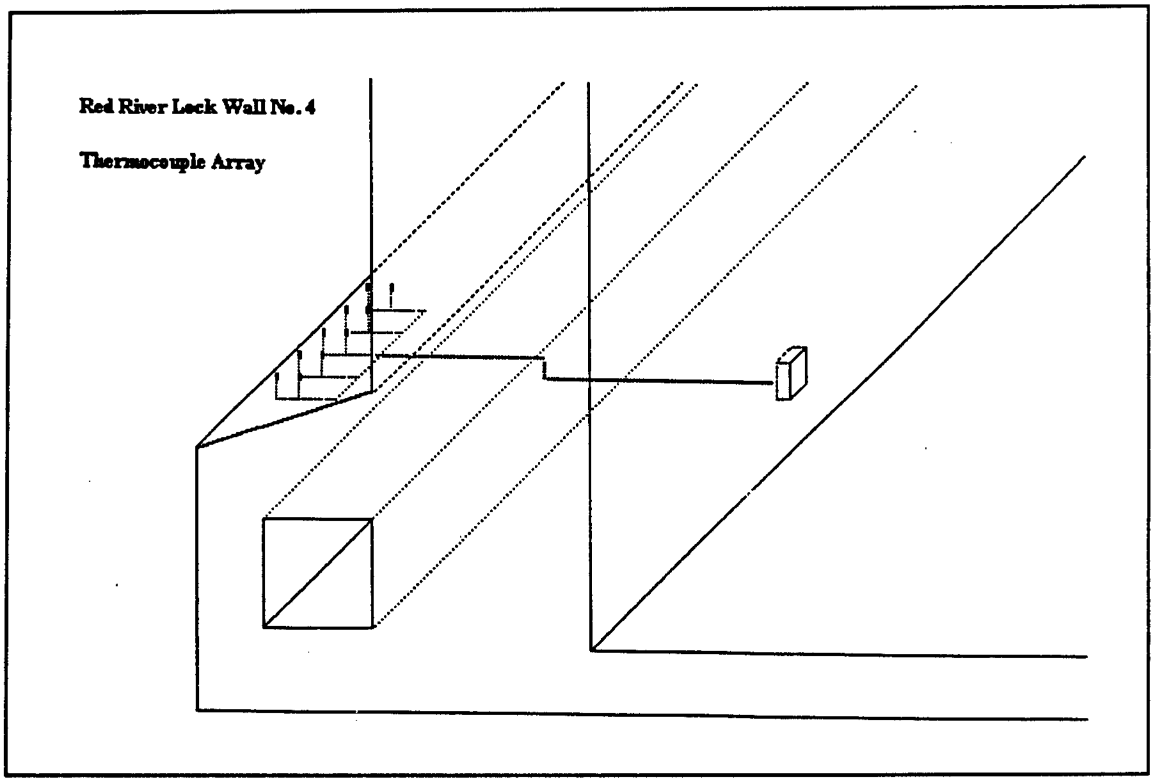
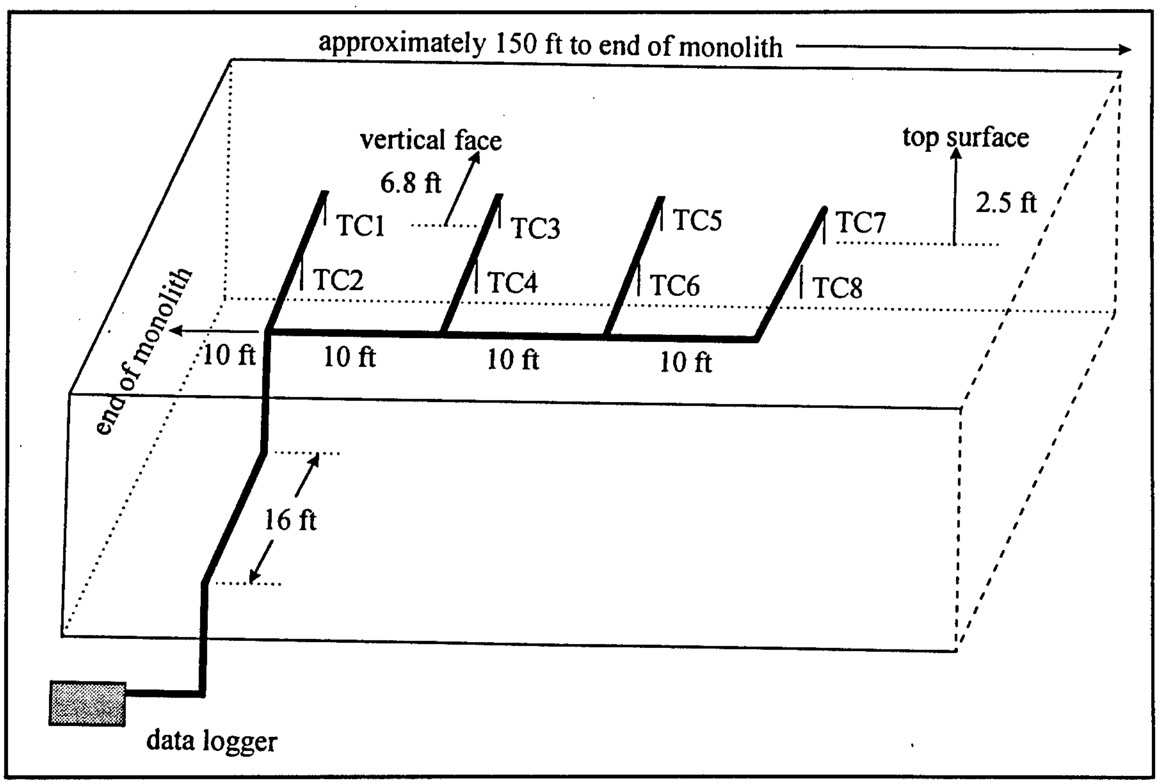


a. Location of thermocouple array in lock wall of L & D 4, Red River



b. Details of thermocouple array in lock wall, as illustrated in part a.

Figure 1. Positioning of thermocouples

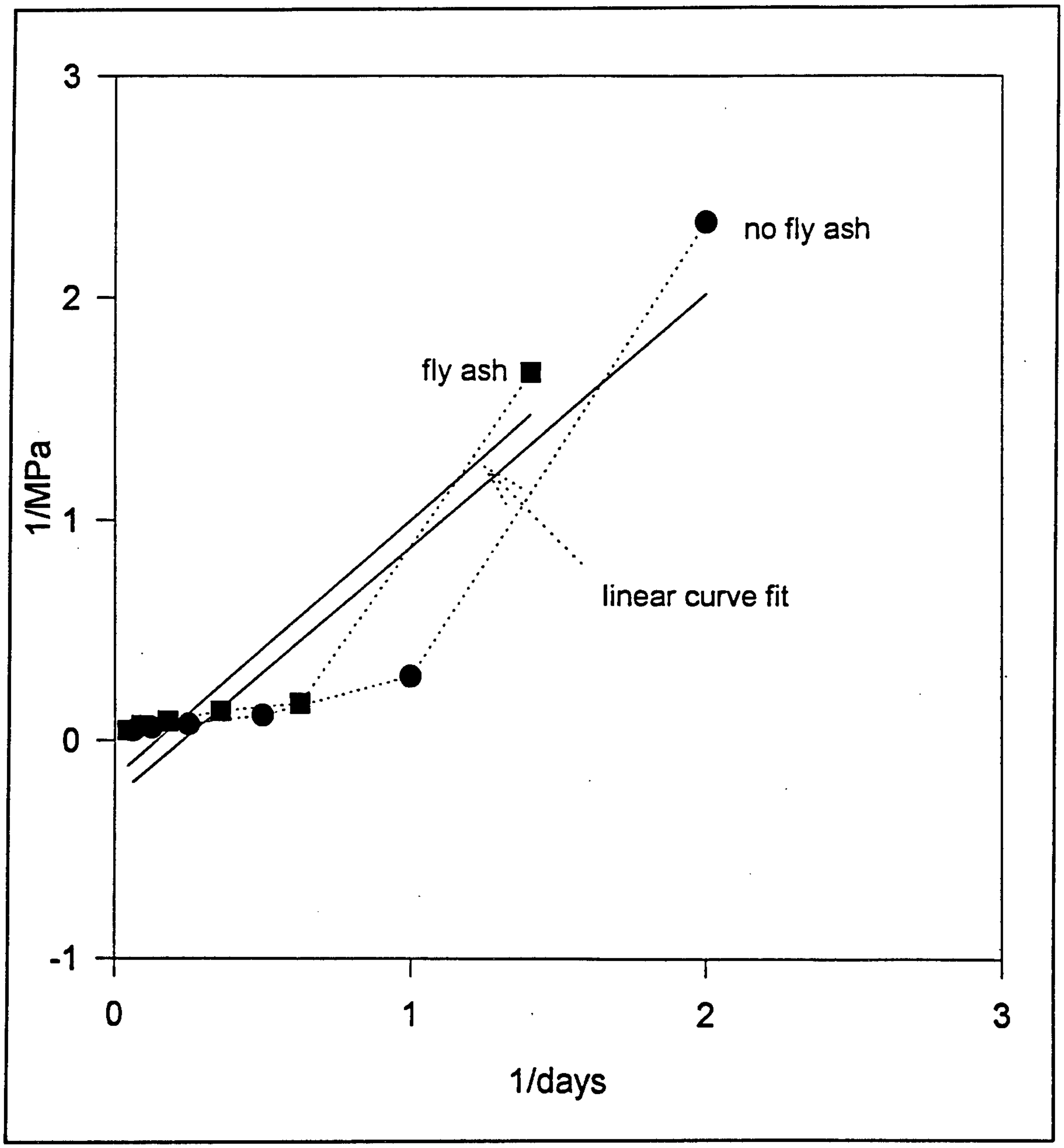


Figure 2. Plot of reciprocal strength vs reciprocal age, illustrating nonlinearity of data

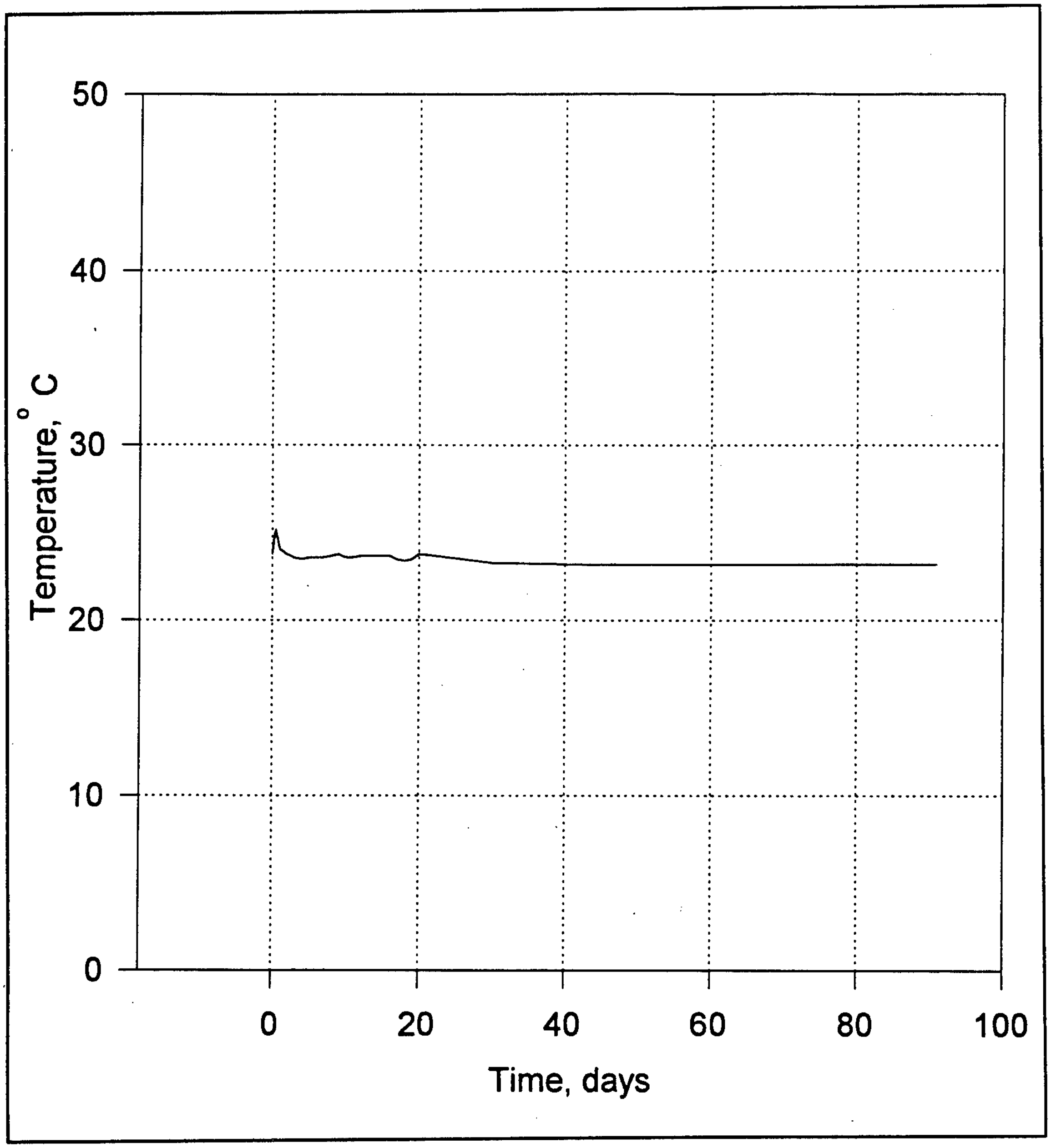


Figure 3. Time-temperature curve for calibration specimens

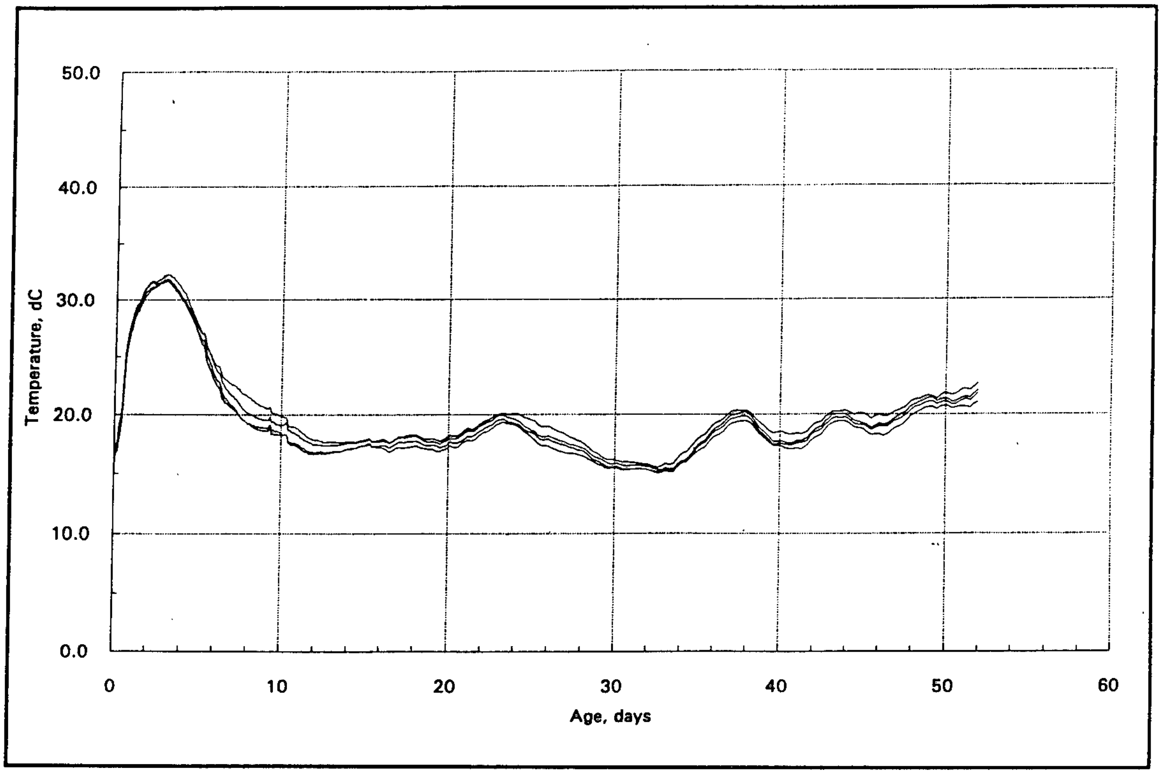


Figure 4. Thermocouple measurements for monolith L-6/7, placed 8 March 1993

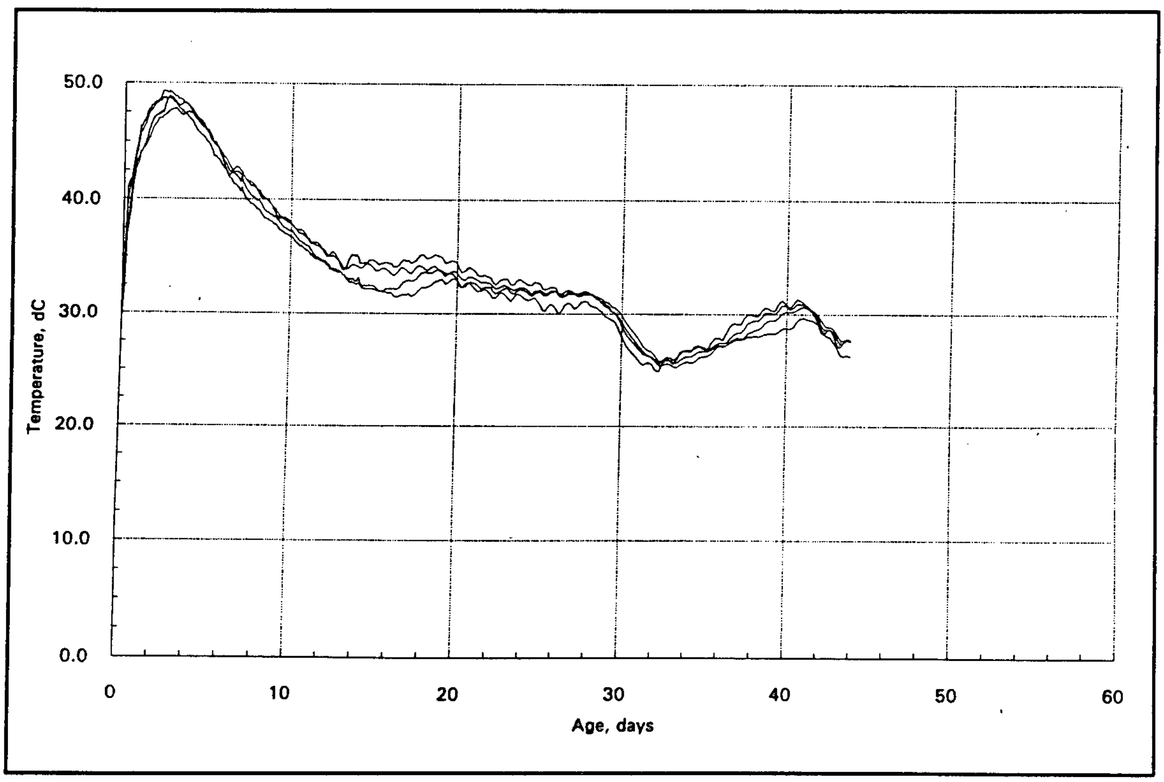


Figure 5. Thermocouple measurements for monolith L-14/15, placed 16 August 1993

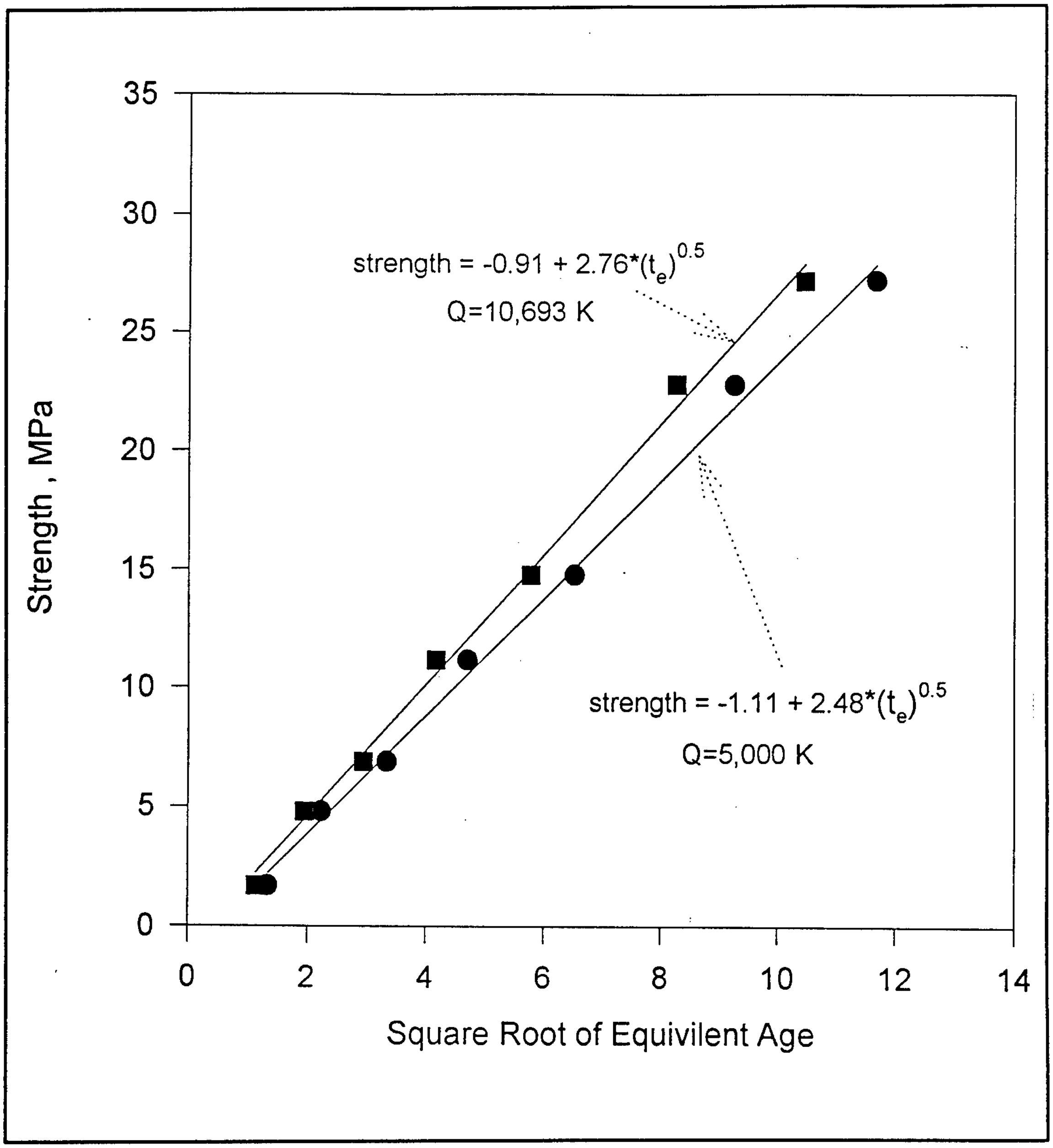


Figure 6. **Calibration curve for equivalent-age calculations**

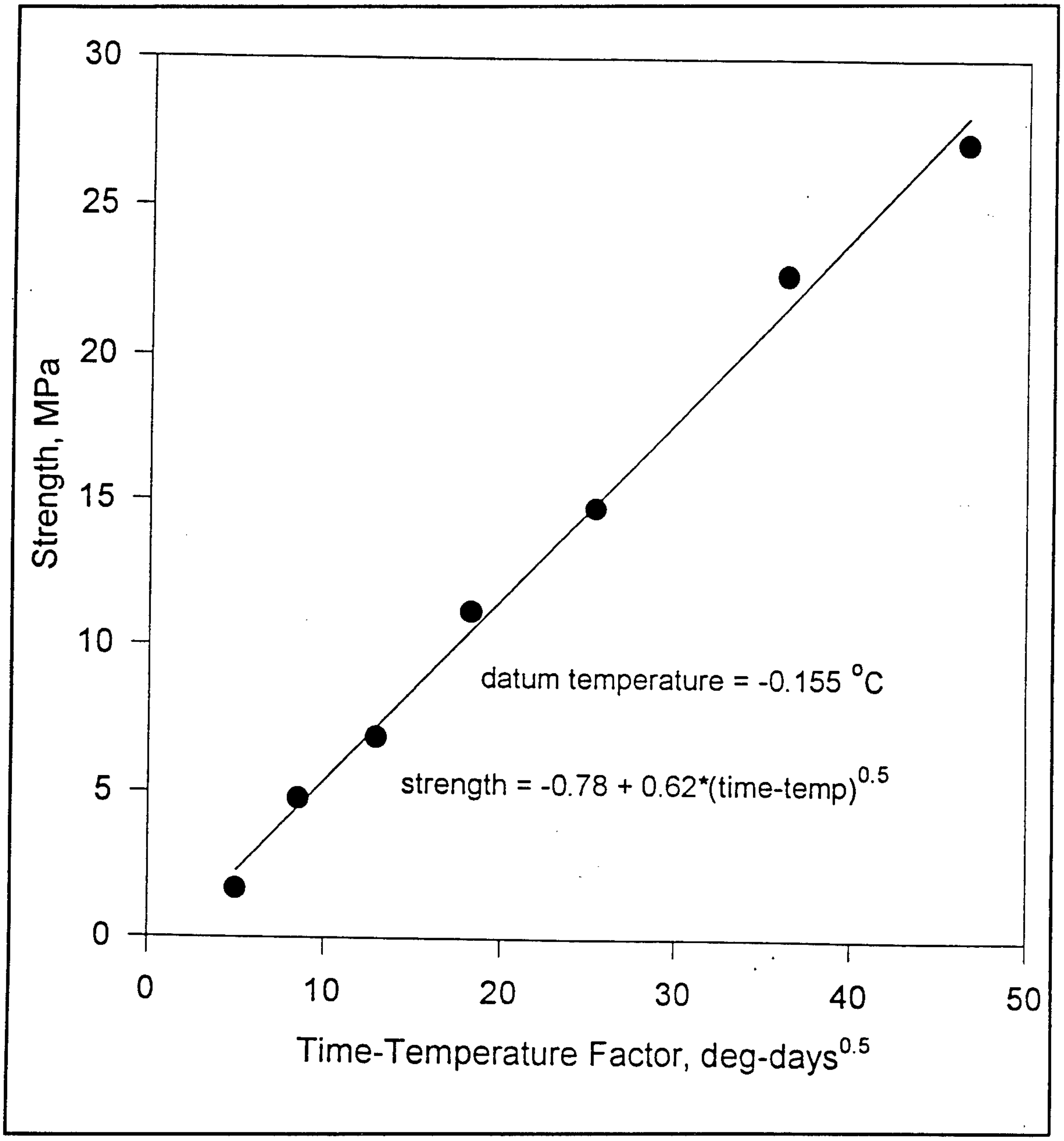


Figure 7. **Calibration curve for time-temperature calculations**

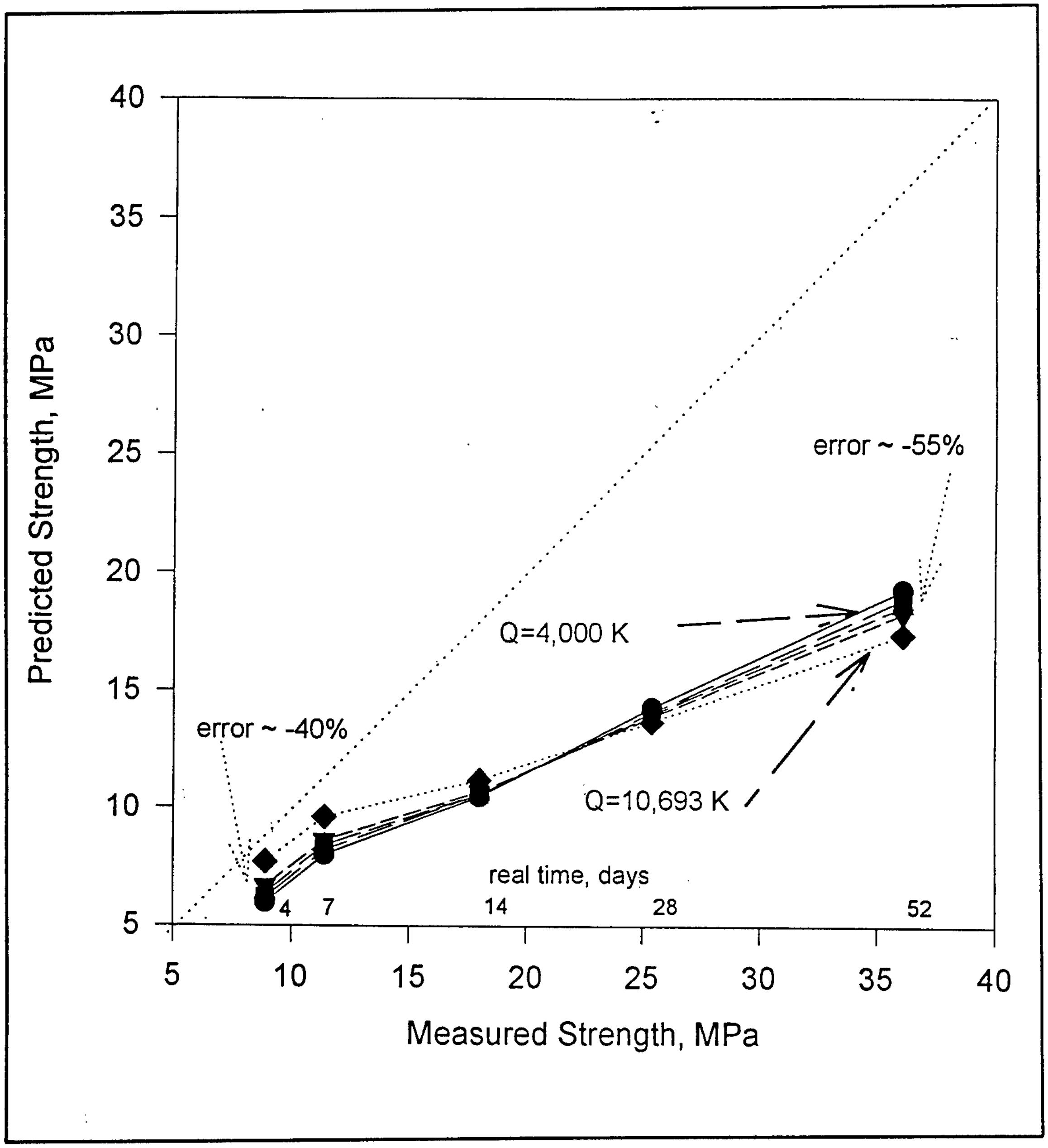


Figure 8. Predicted vs measured strength for L-6/7 using equivalent-age calculations. Q ranges from 4,000 to 10,693 K

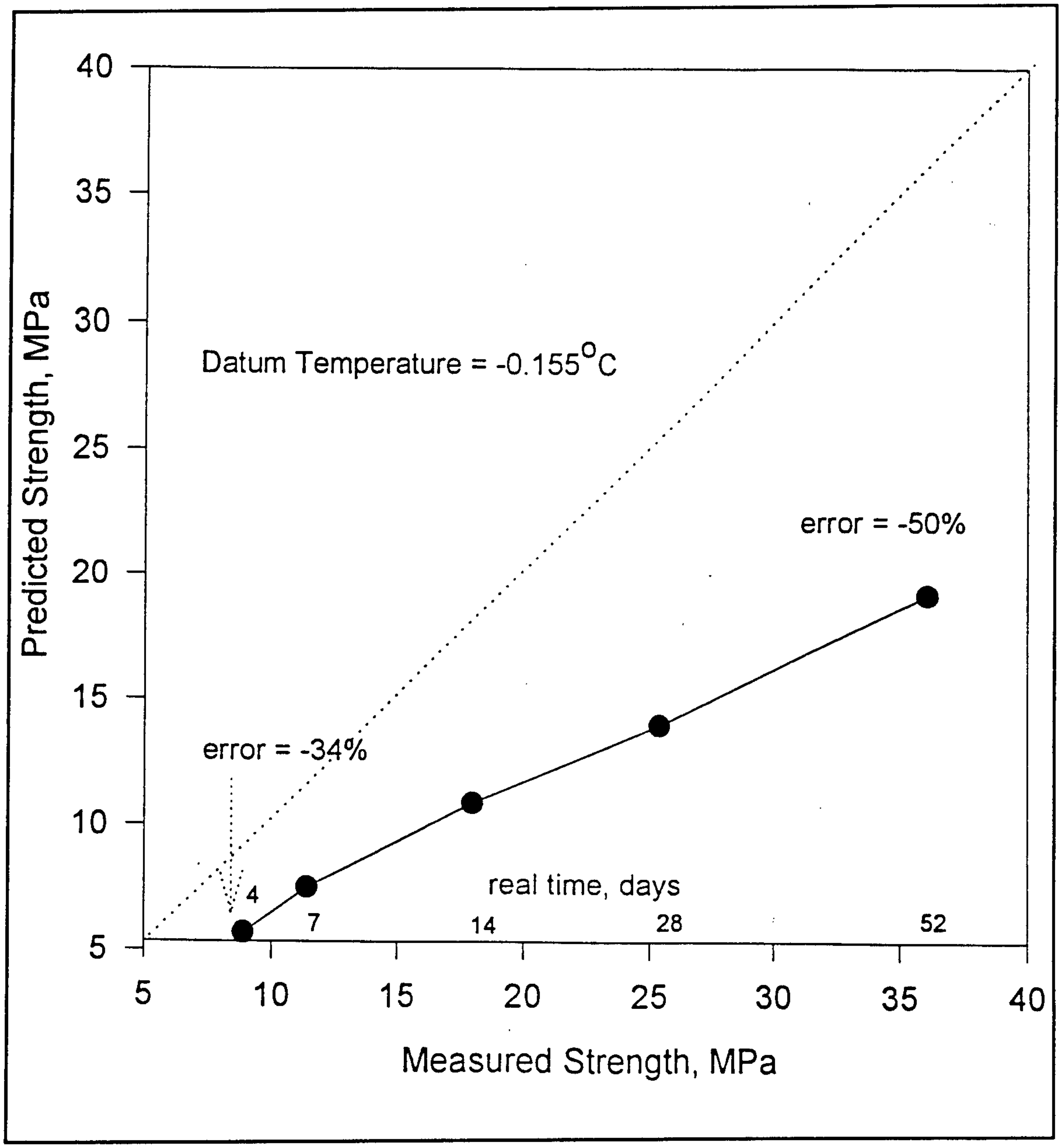


Figure 9. **Predicted vs measured strength for L-6/7 using time-temperature calculations**

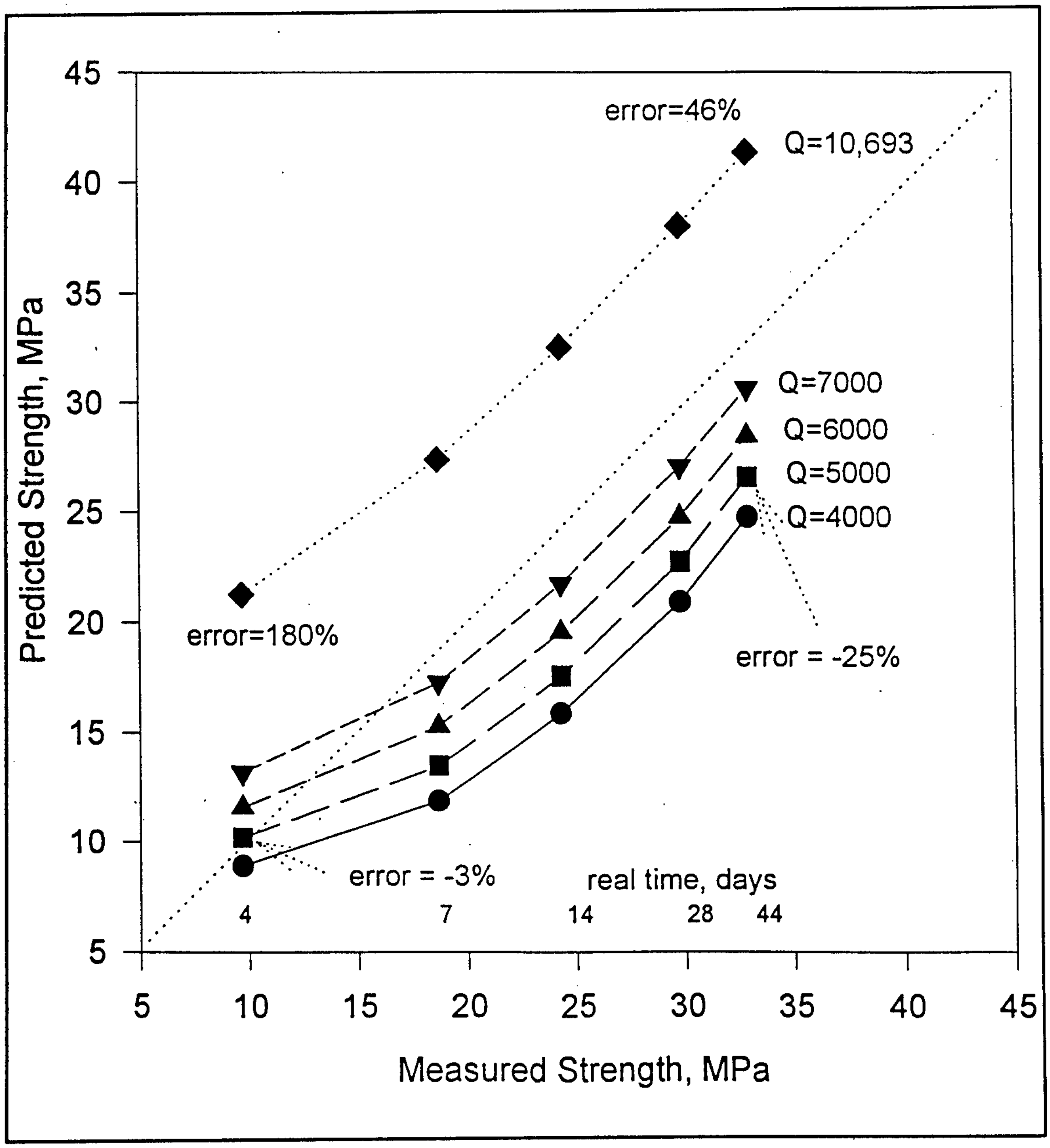


Figure 10. Predicted vs measured strength for L-14/15 using equivalent-age calculations. Q ranges from 4,000 to 10,693 K

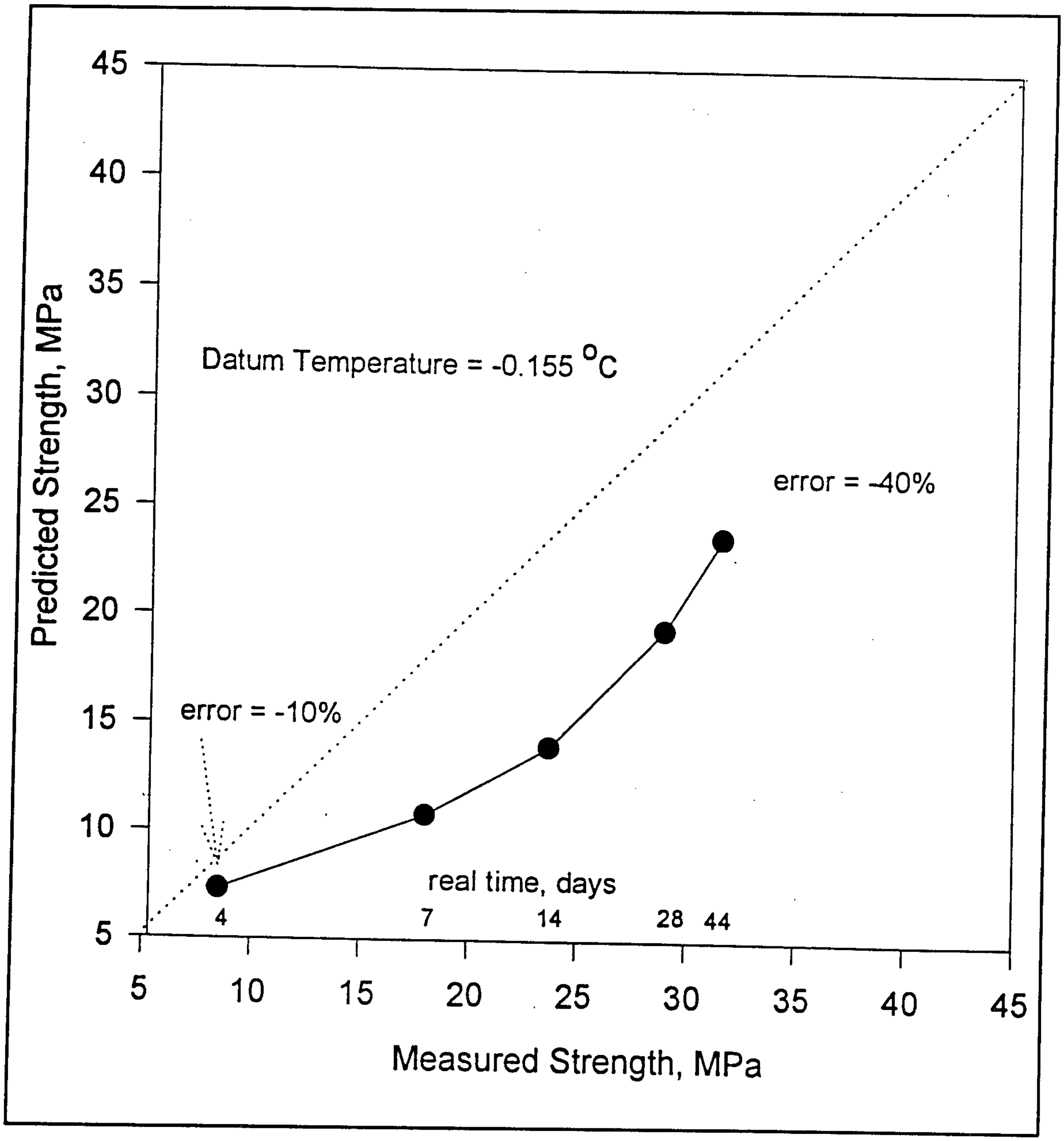


Figure 11. Predicted vs measured strength for L-14/15 using time-temperature calculations

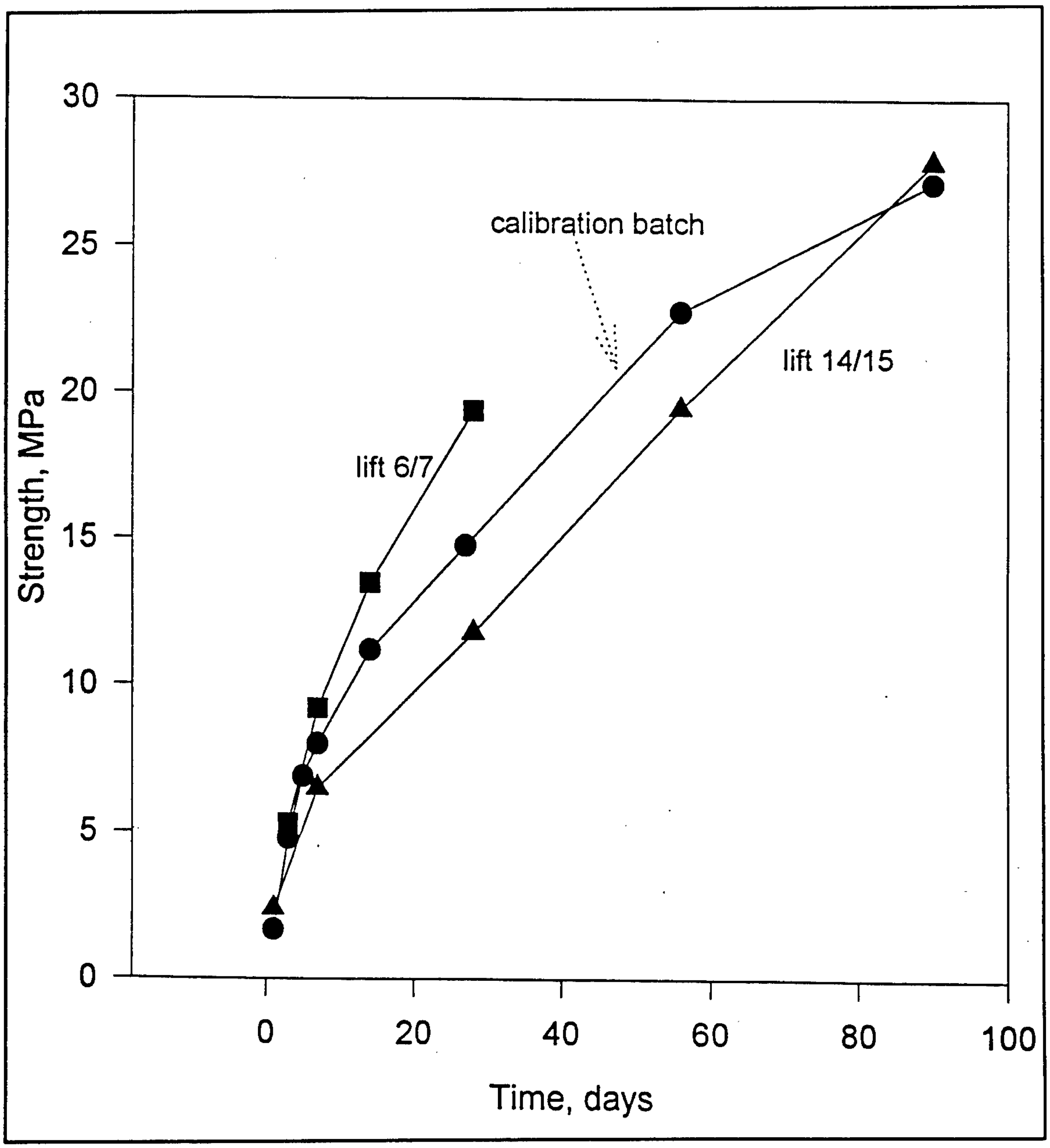


Figure 12. Comparison of strengths of quality control batches with strength of calibration batch

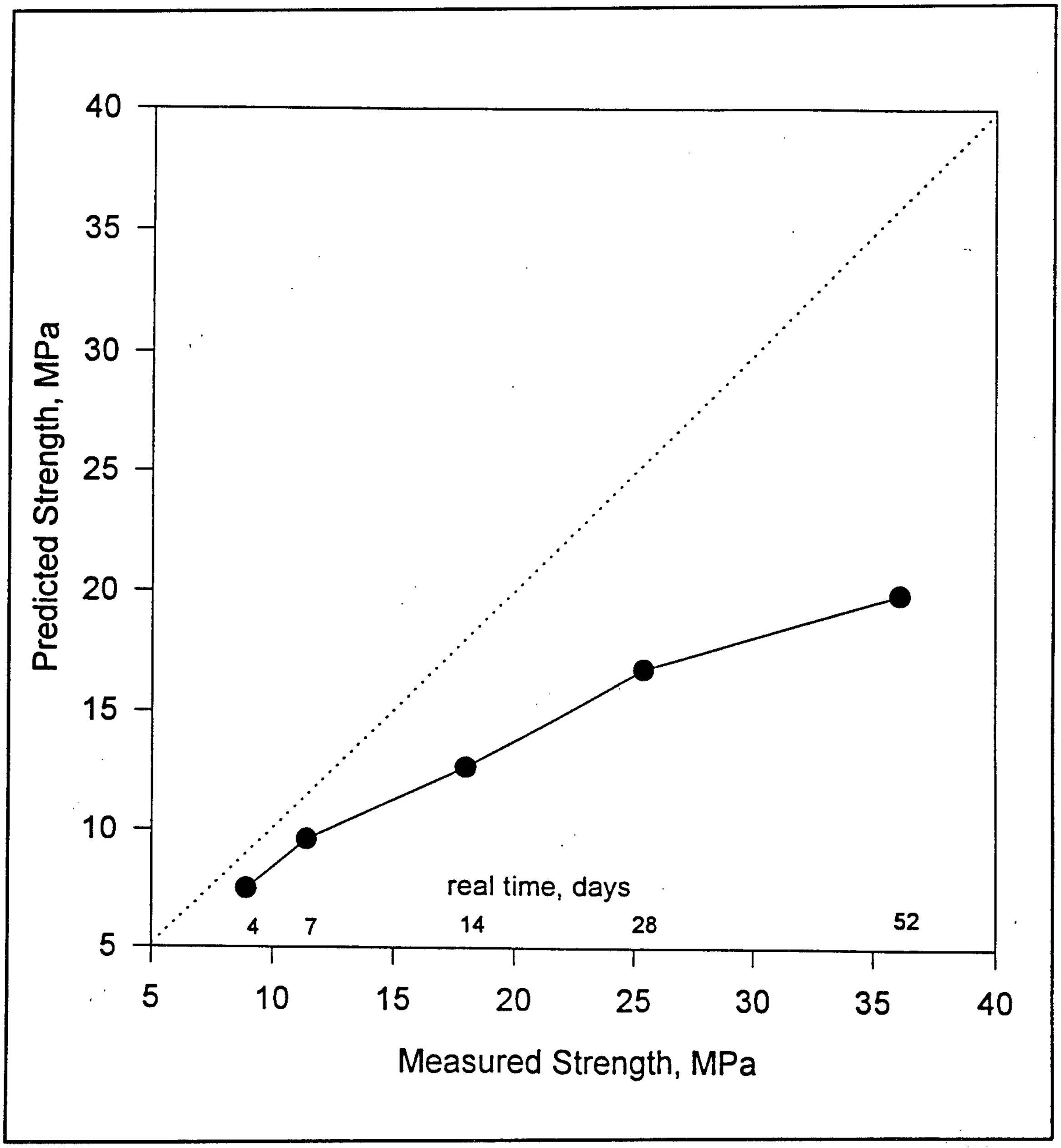


Figure 13. Predicted vs measured strength for L-6/7 using time-temperature calculations and QC data for calibration

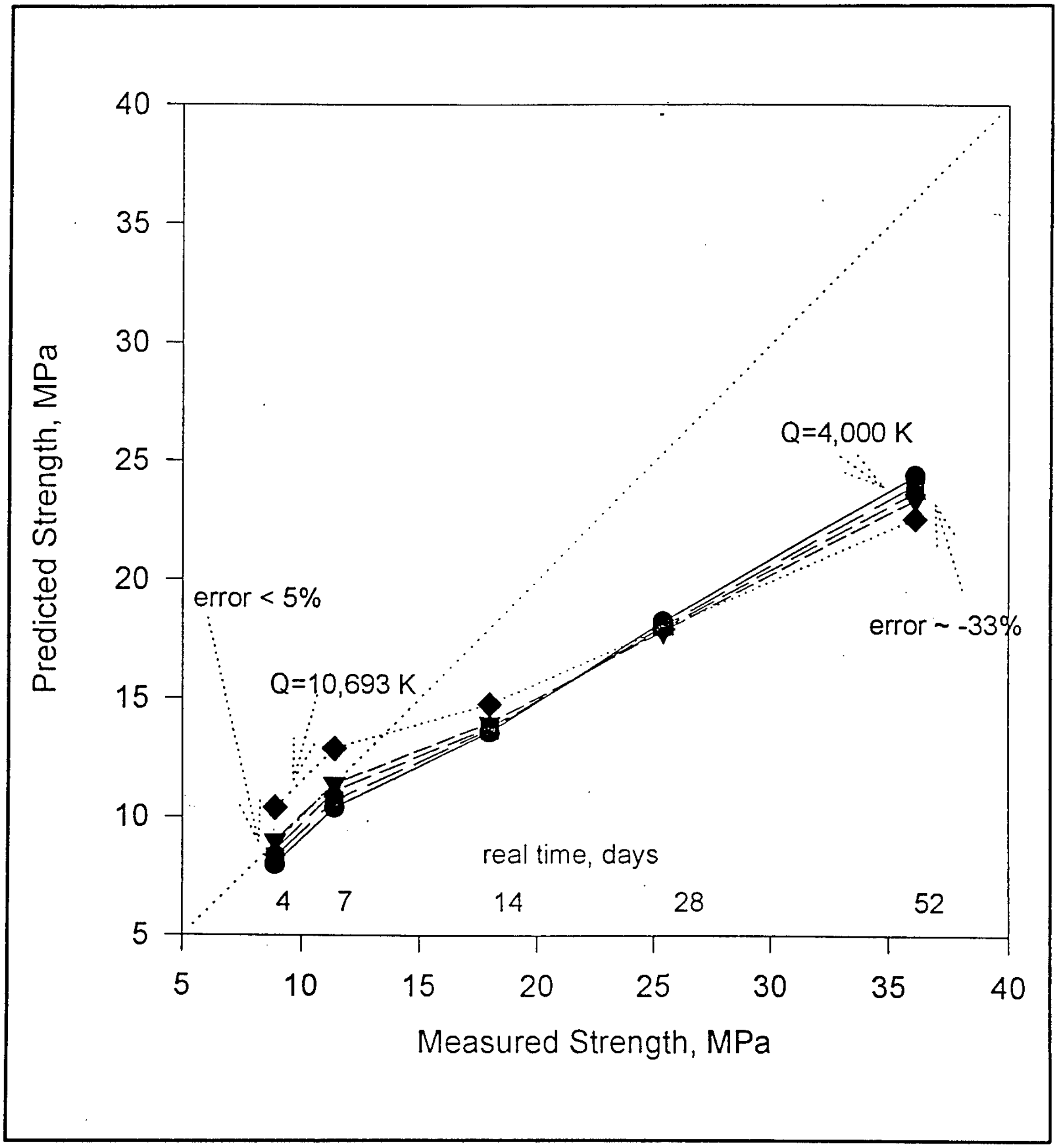


Figure 14. Predicted vs measured strength for L-6/7 using equivalent-age calculations and QC data for calibration

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13. ABSTRACT *(Maximum 200 words)*

Maturity parameters were determined on a mass concrete mixture according to American Society for Testing and Materials (ASTM) C 1074. Time-temperature measurements were made on two concrete placements at Red River Lock and Dam 4 for the first 40 to 60 days after placement. These data were used to predict strengths from the concrete's maturity. The placements were also cored to determine in-place strength. Strength development within the first few days of placing were reasonably well predicted by maturity, but predicted and measured strength differed widely after that. Large uncertainties in the maturity parameters were encountered. Probable causes of the large differences and the large uncertainties are discussed and recommendations made on modifications to the procedure for mass concrete.

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