DEVELOPMENT OF A MOISTURE-MODIFIED MATURITY MODEL
FOR PORTLAND CEMENT CONCRETE PAVEMENTS

Jin-Hoon Jeong[1], Seungwook Lim[2], Dan Zollinger[3]

[1]Dept of Civil Engineering, Inha University
253 Yonghyeon-dong, Nam-gu, Incheon 402-751, South Korea
E-mail: jhj@inha.ac.kr
[2]Division of Research and Innovations, California Dept of Transportation
1101 R St., MS-42, Sacramento, CA 95814, USA
E-mail: s_david_lim@dot.ca.gov
[3]Dept of Civil Engineering, Texas A&M University
Room 503E, CE/TTI Building, College Station, TX 77843-3136, USA
E-mail: d-zollinger@tamu.edu

Abstract. Different mathematical models have been developed to represent the relationship between strength and maturity of concrete. The linear relationship between double logarithmic strength and logarithmic maturity can be implemented in practice to monitor the in-place strength gain and time of opening under variable temperature conditions. Moisture of concrete was considered within the context of existing maturity concepts as a means for improving the predictability of concrete strength. A moisture modification factor developed by Bazant on the basis of Powers' work was modified and included in an existing temperature-based maturity model. An improved relationship between relative strength and maturity was established by using the corrected moisture modification factor.

Keywords: concrete pavement, maturity, strength, non-destructive test, moisture, temperature.

1. Introduction

Curing of newly placed concrete is typically carried out until the concrete strength reaches a minimum required strength. The capability to accurately predict the time of curing (TOC) is a valuable tool in the management of concrete construction scheduling, especially in fast-track construction. Typically, destructive strength testing has been the only method for accurately estimating the TOC. Since the amount of time to prepare and manage specimens for conventional testing sometimes presents coordination difficulties for highway agencies, non-destructive test methods (which use a minimum number of test specimens) should benefit and streamline coordination of quality control measures.

Maturity, a parameter related to the concrete temperature and curing time, has been used as a non-destructive test parameter relative to concrete strength gain. Maturity is a parameter that maintains some proportion to time and

temperature of the concrete aging process and is traditionally calculated from measured concrete temperatures. J. D. McIntosh (1949) and R. W. Nurse (1949) worked out a way to use product of the time and concrete temperature in predicting the concrete strength. Saul suggested a traditional form of maturity model summarising the research results on the principles of steam curing as (Saul 1951):

$$M = \sum_0^t (T - T_0) \cdot \Delta t, \qquad (1)$$

where M – maturity at age t, °C·h; t – age of concrete, h; T – average temperature of concrete during time interval Δt, °C; T_0 – datum temperature below which hydration stops, °C; Δt – time interval, h.

Models of equivalent time originally converted from the Nurse-Saul function (1) has been developed using experimental results (Copeland *et al.* 1960; Weaver, Sadgrove

1971). The equivalent age represents the curing duration at reference temperature which results in the same maturity value as the curing duration at different temperature. The conventional maturity or equivalent age is calculated based upon empirical results. Thus there have been efforts to develop theoretical maturity models to improve accuracy of the conventional models (Gawin *et al.* 2006; Knudsen 1980; 1984). However, the development of the models began from the empirical experiences and included basic assumptions.

The relationship between concrete strength and maturity can be simply used to estimate in-place strength. Different mathematical models (Carino, Tank 1992; Chanvillard, D'Aloia 1997; Kim *et al.* 2001; Pane, Hansen 2002; Tank, Carino 1991; Yi *et al.* 2005) have been developed to represent the strength–maturity relationship which is a formidable task since it depends upon the mixture proportions, concrete curing temperature, and curing time during strength gain. It has been found that a strength–maturity relationship developed on a relative strength (ratio of measured strength to limiting strength) basis can be used for any other temperature conditions as long as the mix proportions remain unchanged (Carino *et al.* 1982). Moreover, on this premise, it is postulated here that a moisture corrected maturity–strength relationship can be used under variable curing conditions.

Concrete hardens with an increase of hydration products formed mainly by the reaction between water and cement. In order to facilitate the hydration process, the moisture in concrete should be maintained at its highest possible level during hydration by preventing evaporation from the surface. Powers (1947) suggested that hydration stops when the relative humidity inside the concrete decreases to 80 %. Thus appropriate moisture retention in concrete during its early stage is important to obtain a high degree of curing. Because the strength of concrete is significantly influenced by the moisture within the concrete, existing maturity concepts should be modified to consider the effect of moisture. The objective of the reported study is to validate the use of moisture corrected maturity developed for a better estimation of in-situ concrete strength.

Table 1. Mix proportions in 1 m^3 of beam specimen concrete

Materials	Proportions
Coarse aggregate (limestone)	1 076 kg
Fine aggregate (natural sand)	749 kg
Cement (type I)	397 kg
Water	127 kg
Water/Cement ratio	0,32
Air entrainment (Paveair 90)	0,1 liters
Water reducer (Pozzolith 300N)	0,3 liters
Superplasticiser (Reobilt 1 000)	3,0 liters
Concrete unit weight	2 352 kg/m^3

2. Maturity calculated by temperature and time

The linear relationship between double natural logarithmic strength and natural logarithmic temperature-based maturity was investigated using laboratory test specimens. Eight beam specimens (2 sets) with the size of 152×152×508 mm were prepared and cured in chambers with 16 °C and 29 °C temperature and 70 % relative humidity for each of the specimens. The mix proportions used for laboratory tests are shown in Table 1. The materials and the water were placed in the chambers 24 h prior to mixing in order to bring their temperature levels to a constant temperature. After placement, thermocouples for temperature and maturity measurements were placed 76 mm into the fresh concrete. The maturity values were calculated by using Eq (1) with a datum temperature value of –10 °C (Saul 1951). The beam specimens cured in the environmental chambers were tested at various times during the 48 h curing period. Flexural strength tests followed Texas Dept of Transportation (TxDOT) procedure *Tex-420-A* (single point loading test) using a hydraulic loading device. Both the maturity and the flexural strength values were recorded at the same points of time.

2.1. Determination of limiting strength

Since the curing temperatures were different for each set of specimens, the limiting flexural strengths were expected to be different for each curing temperature (Carino *et al.* 1982a). Strength development and maturity data were analyzed by a reciprocal method to find the limiting flexural strengths of the specimens as discussed later. Carino (1982a) proposed a reciprocal model by including an offset maturity, M_0 (McIntosh 1949), in the original reciprocal model developed by Kee (1971).

$$\frac{1}{S} = \frac{1}{S_{\text{lim}}} + \frac{1}{A} \cdot \frac{1}{(M - M_0)}, \qquad (2)$$

where S – strength, MPa; S_{lim} – limiting strength, MPa; A – initial slope of strength-maturity curve, MPa·°C^{-1}·h^{-1}; M – maturity, °C·h; M_0 – offset maturity, °C·h.

The offset maturity accounts for the fact that the strength of concrete does not begin to develop until a certain amount of maturity is reached. As seen from Eq (2), there is a linear relationship between the reciprocal of strength and the reciprocal of maturity. This equation indicates that the intercept value is the reciprocal of the limiting strength.

The parameters in Eq (2) have been calculated by performing a regression analysis. The values of M_0 were determined to be 0 °C·h and 180 °C·h respectively when the trend lines of the relationship between the reciprocal of flexural strength and the reciprocal of maturity for the 16 °C and 29 °C specimens had the highest R-square values. Maturity of the concrete was modified by subtracting the

M_0 from the original values. The reciprocal method using the modified maturity could be applied to the data as displayed in Fig 1. The limiting flexural strengths were predicted to be 9,42 MPa and 6,31 MPa respectively for the concrete specimens cured in 16 °C and 29 °C chambers. However, the limiting strength values should be calculated by using a multi-variable regression analysis because the results of Eq (2) are not accurate when the M_0 value is not equal to 0. *Kaleidagraph* or other similar tools can be used to find the parameters of strength–maturity models. In this study, the model (3) suggested by Freiesleben Hansen and Pedersen (1977) showed the best fit among all the valid models referred to previously.

$$S = S_{\lim}e^{-\left[\frac{\tau}{M}\right]^{a}}, \quad (3)$$

where S – strength, MPa; $S_{\lim}$ – limiting strength, MPa; τ – characteristic time constant, °C·h; M – maturity, °C·h; a – shape parameter.

The curve of the strength with the natural logarithmic maturity is shifted to the right or left by changing the value of the time constant while the slope of the curve alters by changing the value of the shape parameter. The regression analysis for the model yielded limiting flexural strength values of 6,59 MPa and 5,63 MPa for specimens cured at 16 °C and 29 °C chambers, respectively. Because of rapid hydration, the early-age strength of the concrete cured at higher temperature is larger than that cured at lower temperature. However, the hydrates produced at higher temperature do not have enough time to be uniformly distributed within pores because of too rapid reaction speed. The non-uniformly distributed hydration products lead to non-uniform pore size which reduces the concrete strength, and the undistributed hydrate lumps impedes hydration of unreacted cements at late age. Therefore, limiting flexural strength of specimens cured at lower temperature was higher than that of the specimens cured at a higher temperature (CCA 1976).

2.2. Linear natural logarithmic relationship between strength and maturity

The previous model of Freisleben Hansen and Pedersen (1977) also accurately represented the linear strength-maturity relationship by using natural logarithmic functions. Eq (3) can be transformed into Eq (4) as:

$$Ln\left(-Ln\left(\frac{S}{S_{\lim}}\right)\right) = aLn\left(\frac{\tau}{M}\right) =$$
$$a\left(Ln\tau - LnM\right) = aLn\tau - aLnM, \quad (4)$$

where S – strength, MPa; $S_{\lim}$ – limiting strength, MPa; τ – characteristic time constant, °C·h; M – maturity, °C·h; a – shape parameter.
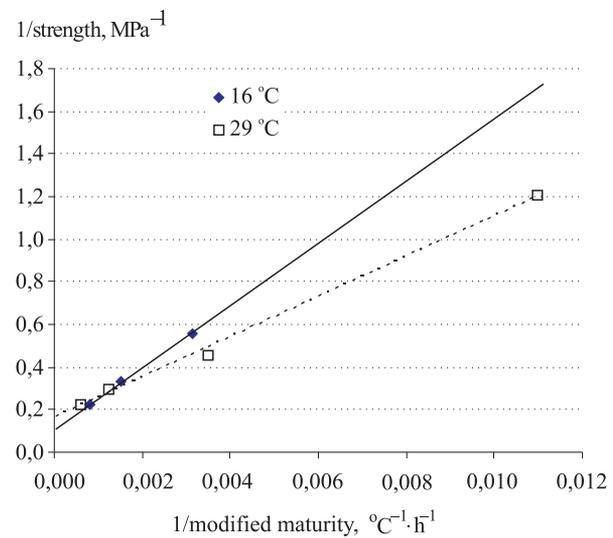
Fig 1. Determination of limiting flexural strength by modified reciprocal method
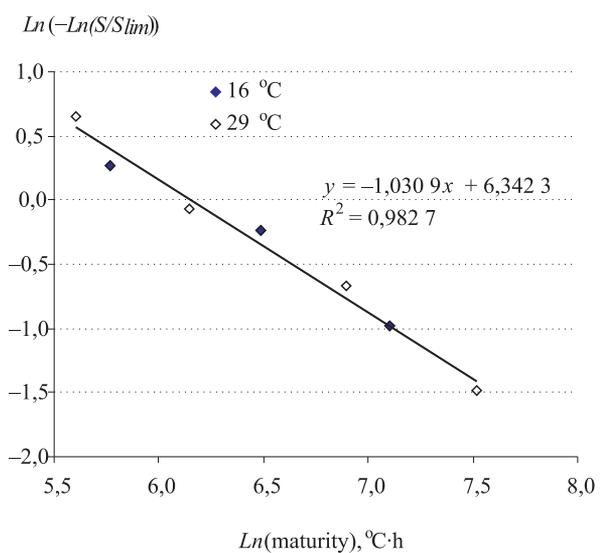
Fig 2. Natural logarithmic relationship between relative strength and maturity for the tested beam specimens

Eq (4) reveals that there is a linear relationship between the double natural logarithm of the relative flexural strength (degree of hydration) and the natural logarithm of the maturity. When limiting strength is calculated, this relationship can be used in predicting the concrete strength on the site where the maturity has been measured.

The linear relationship between double natural logarithm of the relative flexural strength values and the natural logarithm of the maturity for all tested specimens are presented in Fig 2. The linear regression analysis for the combined data points yielded a very high value of R-square. This conclusion supports the hypothesis that the relationship between the relative strength and the maturity is gen-

eral for variable curing temperature and unique for the same mix proportions.

3. Moisture-modified maturity

As previously noted, moisture condition influences strength development of concrete because hydration involves reaction between water and cement. Thus the strength of concrete mixtures consisting of the same mix proportions can be variable with respect to different moisture levels within concrete. This suggests that moisture should be incorporated into the temperature-based maturity concept to better predict the strength gain of concrete under variable moisture conditions.

3.1. Laboratory test program

A laboratory test program was carried out to investigate the moisture-modified maturity of concrete. A cylinder mould with both inside diameter and height of 305 mm was prepared. The mould consisted of 13 mm thick polyvinyl chloride (PVC) wall and an end plate. The mould insulated the concrete against lateral drying effectively causing one-dimensional moisture movement through the top surface. Before placing the concrete, 3 brass casings were threaded into the wall of the mould to support the moisture monitoring sensors at the depths of 25 mm, 76 mm, and 178 mm from top. The sensor locations were determined considering large variations in concrete moisture near top surface exposed to ambient air. Maturity sensors were instrumented inside the mould as shown in Fig 3. Vibrating wire strain gauges were also included in the instrumentation; however, the strain related to the results are not presented in this paper.

Concrete with a low water-cement ratio was placed in the mould slowly and then compacted carefully according to *ASTM C 192: Standard practice for making and curing concrete test specimens in the laboratory* (American Society for Testing and Materials) to avoid settlement of large particles of aggregate and subsequent bleeding. As shown in Table 2, water to cementation materials ratio of 0,32 was used and the unit weight of concrete was determined by *ASTM C 29: Standard test method for unit weight and voids aggregates* (American Society for Testing and Materials) and was found to be 2 439 kg/m³. Immediately after the placement, the specimen was moved to an environmental chamber set at 60 °C and 15 % relative humidity. Three and a half hours after placement, three temperature and moisture monitoring sensors manufactured by ATEK (Wang 2000) were placed in the previously noted positions to measure temperature and relative humidity of the concrete. The relative humidity sensors which have ±2 % error range operate on chilled mirror technology, which measure the dry bulb and the dew point temperature of inside brass casings. Access holes in the casings allowed the vapour pressure of the concrete to equilibrate inside the casings. Conventional maturity of the concrete specimen was also measured by a maturity meter.

Concrete splitting tensile strength was also determined at 25 mm, 76 mm, and 178 mm from the top surface using

Table 2. Mix proportions in 1 m³ of cylinder specimen concrete

Materials	Proportions
Coarse aggregate (sandstone)	1 076 kg
Fine aggregate (natural sand)	869 kg
Cement (type I)	281 kg
Fly ash (type C)	90 kg
Water	120 kg
Water/Cementitious materials ratio	0,32
Superplastisiser (Reobilt 1 000)	2,7 litres
Concrete unit weight	2 439 kg/m³
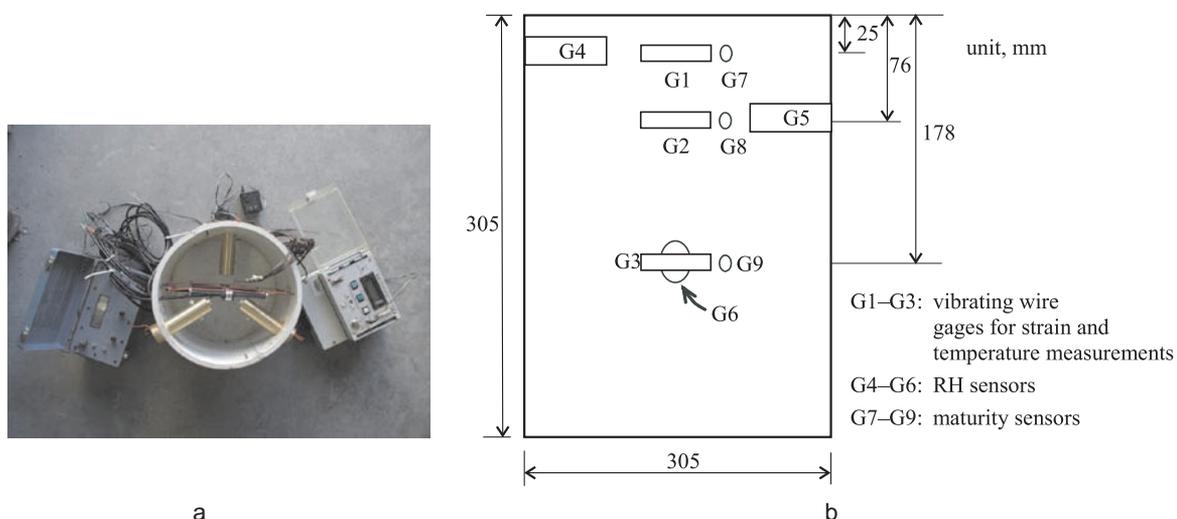
Fig 3. Instrumented 305×305 mm PVC cylinder mould: a – picture of data acquisition devices; b – schematic of test setup

Fig 4. Instrumented 152×305 mm standard cylinder mould

standard 152×305 mm cylinder specimens. The specimens were also cured in the 60 °C and 15 % chamber similar to the 305×305 mm cylinder specimen. As shown in Fig 4, the temperature was also recorded by thermometers to correlate to the maturity of the 305×305 mm cylinder specimen. The standard cylinder specimens were demoulded 3 days, 9 days, and 15 days after placement and subsequently cut into 3 pieces with 51 mm thickness at 25 mm, 76 mm, and 178 mm from top surface (at the same depths as the moisture sensors). Although the 51 mm of the thickness was not adequate for accurate splitting tensile strength, good trends of relative splitting tensile strengths were expected because the pieces had the same size. Splitting tensile strength tests were conducted in accordance with *ASTM C 496: Standard test method for splitting tensile strength of cylindrical concrete specimens* (American Society for Testing and Materials) for each piece of the specimens immediately after cutting.

3.2. Temperature and moisture of concrete

Temperature, relative humidity, and conventional maturity of the 305×305 mm cylinder specimen were determined over a 21-day curing period at each depth. The concrete temperature at the time of placement was 23 °C and reached a maximum temperature of 62 °C approx 12 h after placement, as shown in Fig 5a. Subsequently, the temperature of the specimen is stabilised near 60 °C. The trend of maturity with curing time was almost linear because of the continuously stabilised temperature of the specimen as shown in Fig 5b. Because of a low relative humidity in the environmental chamber, the relative humidity near top surface of the specimen decreased over the 21-day period as shown in Fig 5c due to a high level of evaporation. The relative humidity reached 53 %, 82 %, and 91 % at 25 mm, 76 mm, and 178 mm of depths during 21 days. As shown in Fig 5, the relative humidity was rarely uniform and far from equilibrium during the test period.

a

b

c
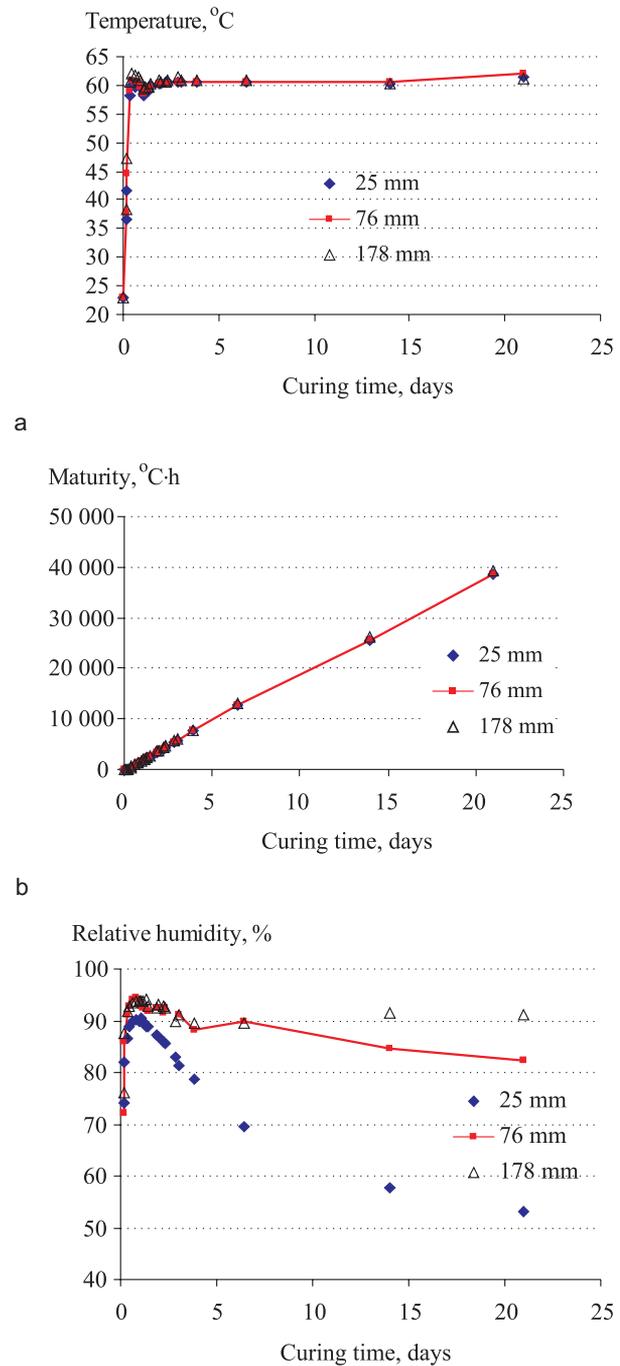
Fig 5. Temperature, moisture, and maturity histories at different depths of the 305×305 mm cylinder specimens:
a – temperature; b – maturity; c – moisture

3.3. Strength of concrete at different moisture levels

Concrete splitting tensile strength at 25 mm, 76 mm, and 178 mm from the top surface of the strength specimens were determined by splitting tensile testing of 51 mm slices cut at the depths. The splitting tensile strength data was correlated to the moisture and the maturity data obtained from the moisture specimen placed in PVC mould. At the same conventional maturity, splitting tensile strengths of each

Split tensile strength, MPa
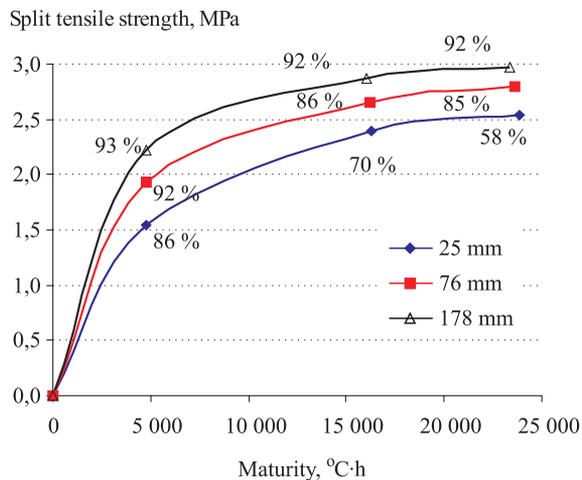
Fig 6. Measured splitting tensile strengths and relative humidity with maturity at different depths of specimens

$Ln(-Ln(S/S_{lim}))$

$y = -0,8574x + 6,5735$
$R^2 = 0,8205$

Standard error
= 1,438 (intercept)
= 0,152 (slope)

◆ 25 mm
■ 76 mm
△ 178 mm

$Ln(M)$, °C·h
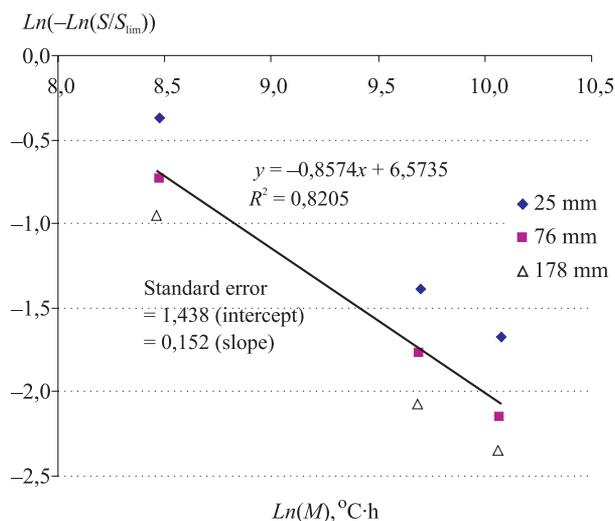
Fig 7. Natural logarithmic relationship between relative splitting tensile strength and maturity at different depths of specimens

Moisture parameter

—— Current study
- - - Bazant

Relative humidity, %
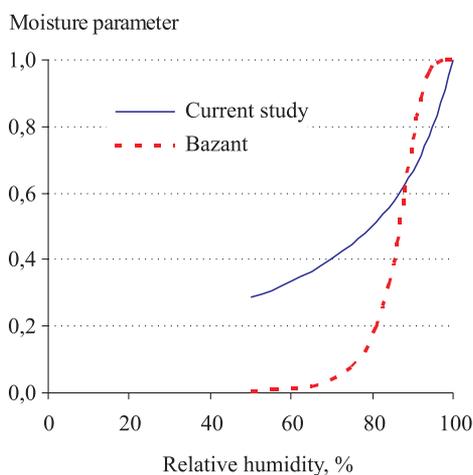
Fig 8. Comparison of corrected moisture modification factor with the existing one (Bazant 1969)

piece cut from different depths of the standard cylinder specimens are separated (Fig 6) due to the different moisture levels (Fig 5c). This implies that temperature-based maturity concepts have shortcomings in predicting concrete strength when curing under different moisture conditions.

By using the linear relationship between reciprocal of splitting tensile strength and the reciprocal of maturity, limiting splitting tensile strength was predicted as 3,07 MPa, 3,14 MPa, and 3,26 MPa for the segments cut at 25 mm, 76 mm, and 178 mm from top surface, respectively. The trends of double natural logarithm of the relative splitting tensile strength of each segment against natural logarithm of the maturity are different by depths as presented in Fig 7, although they were from the same mix proportions. Because of the different trends of each segment, standard errors of the intercept and the slope of the trend line made from all data points were relatively high, as shown in Fig 7. This fact implies that the moisture, one of the most important factors of hydration of concrete, should be incorporated into the existing temperature-based maturity for a better prediction of strengths of concrete under variable moisture conditions.

3.4. Moisture modification factor

A moisture modification factor shown in Eq (5) was originally made for numerical computation of moisture effect on equivalent age (Bazant 1969; 1970) based on the observation of concrete moisture (Powers 1947).

$$\beta_H = \left[1 + \left(7,5 - 7,5H\right)^4\right]^{-1}, \qquad (5)$$

where β_H – the moisture modification factor; H – relative humidity of concrete, unit of decimal points.

The value of the moisture modification factor decreases as the relative humidity of concrete decreases as shown in Fig 8. The moisture modification factor indirectly represents the influence of moisture upon the rate of hydration reaction. Therefore the strength of concrete at a level of moisture can be adjusted by the moisture modification factor. An example being the adjustment of the Nurse–Saul maturity function [Eq (1)] by incorporating the moisture modification factor is:

$$M_H = \beta_H \times \sum_0^t \left(T - T_0\right) \cdot \Delta t = \frac{\sum_0^t \left(T - T_0\right) \cdot \Delta t}{1 + \left(7,5 - 7,5H\right)^4}, \qquad (6)$$

where M_H – the moisture-modified maturity, °C·h; β_H – the moisture modification factor; t – age of concrete, h; T – average temperature of concrete during time interval Δt, °C; T_0 – datum temperature below which hydration stops, °C; Δt – time interval, h; H – relative humidity of concrete, unit of decimal point.

The relationship between the double natural logarithm

of the relative splitting tensile strength and natural logarithm of the moisture-modified maturity, presented in Fig 9, shows an improved linear trend comparing to the *R*-square value and standard errors of the trend line in Fig 7. The improved linear relationship implies that the moisture-modified maturity is useful in prediction of concrete strengths under variable curing conditions.

The trend line in Fig 9 was made only for the data points with relative humidity over 80 %. This is because the moisture modification factor was originally modelled under the suggestion that hydration completely stopped, when the relative humidity of the drying concrete decreased to approx 80 %. Hence, there exist two erratic points which are associated with concrete tested at a relative humidity below 80 %. Nonetheless, the splitting tensile strength increased steadily in spite of the measured relative humidity below 80 % as shown in Fig 6. This indicates a basis to suggest a modification of the model coefficients originally used by Bazant (1960, 1970). The model of moisture-modified maturity (Eq (6)) was improved by modifying the coefficients in moisture modification factor shown in Eq (7).

$$\beta'_H = \left[1 + (a - aH)^b\right]^{-1}, \qquad (7)$$

where β'_H – the corrected moisture modification factor; *a*, *b* – coefficients.

The moisture-modified maturity is re-formulized by using the previous corrected moisture modification factor which uses 5,0 and 1,0 as its coefficients, *a* and *b*.

$$M'_H = \beta'_H \cdot \sum_{0}^{t}(T - T_0)\cdot \Delta t = \frac{\sum_{0}^{t}(T - T_0)\cdot \Delta t}{1 + (5 - 5H)}, \qquad (8)$$

where M'_H – the corrected moisture-modified maturity, °C·h; β'_H – the corrected moisture modification factor; age of concrete, h; *T* – average temperature of concrete during time interval Δt, °C; T_0 – datum temperature below which hydration stops, °C; Δt – time interval, h; *H* – relative humidity of concrete, unit of decimal point.

Fig 8 shows that the corrected moisture modification factor steadily drops with a decrease of the concrete relative humidity, while the previous moisture modification factor radically drops between 80 % and 90 % of concrete relative humidity. In Fig 8, trend of the corrected moisture modification factor is compared to that of the previous moisture modification factor over 50 % of concrete relative humidity because the lowest concrete relative humidity during the laboratory tests was 58 % at 25 mm depth of the specimen 21 days after placement. However, predicting the factor over 50 % of concrete relative humidity is enough because the concrete specimen used in the laboratory tests was cured in extremely low relative humidity and high temperature conditions during the tests. In addition,
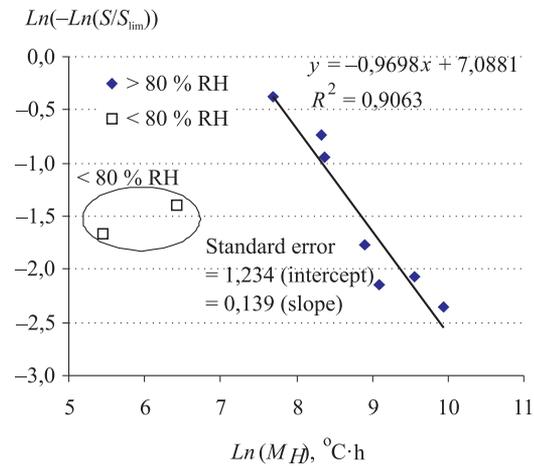
Fig 9. Natural logarithmic relationship between relative splitting strength and moisture-modified maturity at all depths of specimens
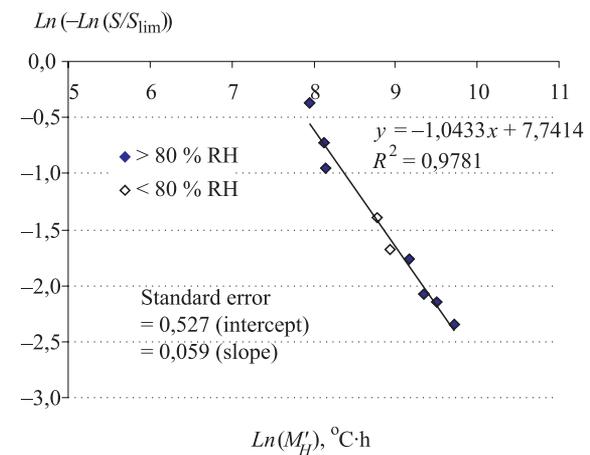
Fig 10. Improved natural logarithmic relationship between relative splitting tensile strength and corrected moisture-modified maturity at all depths of specimens

21 days of curing is sufficient for modelling the moisture modification factor because maturity is necessary to estimate the strength development of early-age concrete.

The improved natural logarithmic relationship between relative splitting tensile strength and corrected moisture-modified maturity is shown in Fig 10. The *R*-square value and the standard errors of the trend line shown in the chart were made from all data points in Fig 10.

3.5. Modelling validation

Another laboratory test was conducted following the same procedure and instrumentation as the previous one to demonstrate the validity of the proposed moisture-modified maturity model [Eq (8)]. To verify the effectiveness of the moisture-modified maturity model for concrete with other mixture proportions, test specimens were made with a different mixture design from the previous one as shown in

Table 3. Mix proportions in 1 m^3 of cylinder specimen concrete for modelling validation

Materials	Proportions
Coarse aggregate (limestone)	707 kg
Intermediate aggregate (limestone)	423 kg
Fine aggregate (natural sand)	531 kg
Cement (type I)	243 kg
Fly ash (type C)	81 kg
Water	136 kg
Water/Cementation materials ratio	0,42
Air entrainment (Paveair 90)	0,2 liters
Concrete unit weight	2 121 kg/m^3
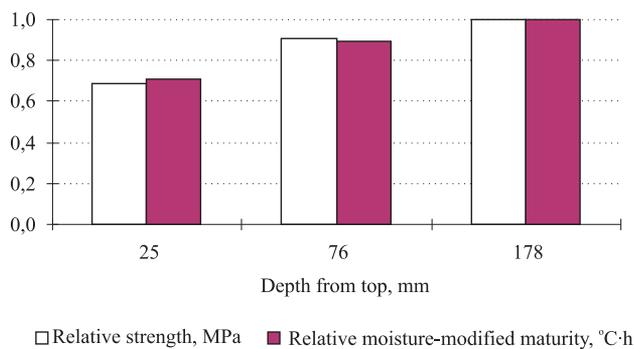
Fig 11. Comparison of moisture-modified maturity ratios with splitting tensile strength ratios at different depths (1,0 at 178 mm depth)

Table 3. A water cement ratio of 0,42 was used and the unit weight of concrete was found to be 2 121 kg/m^3. The specimen was cured in an environmental chamber with 40 °C temperature and 30 % relative humidity for 28 days causing the relative humidity of the specimen to drop below 80 %.

Since the concrete temperature stabilised to the room temperature (40 °C) approx 2 days after placement, the specimen had almost the same temperature-based maturity values (34 900 °C·h) throughout its depth in 28 days. However, moisture-modified maturity values of 11 060 °C·h, 13 850 °C·h, and 15 550 °C·h at 25 mm, 76 mm and 178 mm depths were calculated by Eq (8) and relative humidity of 57 %, 70 %, and 75 % measured at that time. Measured splitting tensile strengths of 2,52 MPa, 3,34 MPa, and 3,68 MPa respectively were related to the relevant maturities. Ratios of the moisture-modified maturity values at 25 mm and 76 mm to that at 178 mm was almost the same as ratios of the splitting tensile strength values at 25 mm, 76 mm to that at 178 mm depth, as shown in Fig 11. This indicates that conventional temperature-based maturity was improved by considering the moisture effect on concrete strength.

4. Conclusions

An attempt was made to verify that the linear relationship between the double natural logarithm of relative strength and the natural logarithm of the maturity is general for the variable curing temperature and unique for the same mix proportions. Hence, it was suggested that the strength-maturity relationship can be applied to a given mixture design to predict the strength over time under a variable curing temperature.

Moisture loss of the concrete results in a significant quality loss. To consider the effect of moisture on the strength–maturity relationship, the Nurse–Saul maturity model was modified by including a moisture modification factor. The moisture modification factor was originally developed based on the suggestion that hydration stops when the relative humidity of the drying concrete decreases by approx 80 %. However, it was observed that concrete splitting tensile strength steadily increased below 80 % of relative humidity and, hence, the moisture modification factor was corrected. This was used to improve the linear natural logarithmic relationship between relative splitting tensile strength and maturity.

The prediction of the maturity of field concrete with temperature and relative humidity varying with time and depth can be accomplished by well-known computer programs solving heat transfer and moisture diffusion equations. To consider the irregular moisture variation due to changeable environmental conditions in the field, a different moisture modification factor which accounts for the moisture history of concrete should be utilised.

References

Bazant, Z. P. 1969. *Thermodynamic theory of concrete deformation at variable temperature and humidity*. Report No 69-11. Division of Structural Engineering and Structural Mechanics, University of California at Berkeley. 100 p.

Bazant, Z. P. 1970. Constitutive equation for concrete creep and shrinkage based on thermodynamics of multiphase systems, *Materials and Structures*, RILEM, 3(13): 3–36.

Carino, N. J.; Lew, H. S.; Volz, C. K. 1982a. Early-age temperature effects on concrete strength prediction by the maturity method, *Journal of American Concrete Institute* 80(2): 93–101.

Carino, N. J. 1982b. Maturity functions for concrete, in *Proc of the RILEM International Conference on Concrete at Early Ages.* Ecole Nationale des Ponts et Chausses, Paris, France, 111–115.

Carino, N. J; Tank, R. C. 1992. Maturity functions for concretes made with various cements and admixtures, *ACI Materials Journal* 89(2): 188–196.

CCA. 1976. *Research and development – research on materials*. Annual report. Cement and Concrete Association. 14–19.

Chanvillard, G.; D'Aloia, L. 1997. Strength estimation at early ages: modification of the method of equivalent age, *ACI Materials Journal* 94(6): 520–530.

Copeland, L. E.; Kantro, D. L.; Verbeck, G. 1960. Chemistry of hydration of portland cement, in *Proc of the 4th International Symposium on the Chemistry of Cement* 1, Washington DC, 429–468.

Freiesleben-Hansen, P.; Pedersen, E. J. 1977. Maturity computer for controlled curing and hardening of concrete, *Nordisk Betong*, Stockholm, Sweden, 464–470.

Gawin, D.; Pesavento, F.; Schrefler, B. A. 2006. Hygro-thermo-chemo-mechanical modelling of concrete at early ages and beyond. Part I: Hydration and hygro-thermal phenomena, *International Journal for Numerical Methods in Engineering* 67(3): 299–331.

Kee, C. F. 1971. Relation between strength and maturity of concrete, *ACI Journal* 68(3): 196–203.

Kim, J. K.; Han, S. H.; Lee, K. M. 2001. Estimation of compressive strength by a new apparent activation energy function, *Cement and Concrete Research* 31(2): 217–225.

Knudsen, T. 1980. On particle size distribution in cement hydration, in *Proc of the 7th International Congress on the Chemistry of Cement*, Edition Septima II, Paris, 170–175.

Knudsen, T. 1984. The dispersion model for hydration of portland cement: I. General concepts, *Cement and Concrete Research* 14: 622–630.

McIntosh, J. D. 1949. Electrical curing of concrete, *Magazine of Concrete Research* 1(1): 21–28.

Nurse, R. W. 1949. Steam curing of concrete, *Magazine of Concrete Research* 1(2): 79–88.

Pane, I.; Hansen, W. 2002. Concrete hydration and mechanical properties under non-isothermal conditions, *ACI Materials Journal* 99(6): 534–542.

Powers, T. C. 1947. A discussion of cement hydration in relation to the curing of concrete, in *Proc of the Highway Research Board* No 27, Highway Research Board, Washington DC, 178–188.

Saul, A. G. A. 1951. Principles underlying the steam curing of concrete at atmospheric pressure, *Magazine of Concrete Research* 2(6): 127–140.

Tank, R. C.; Carino, N. J. 1991. Rate-constant functions for strength development of concrete, *ACI Materials Journal* 88(1): 74–83.

Wang, L. 2000. *Early age characterization of concrete temperature and moisture parameters*. PhD dissertation, Texas A&M University, College Station, TX. 222 p.

Weaver, J.; Sadgrove, B. M. 1971. *Striking times of formwork-tables of curing periods to achieve given strengths*. Report No 36, Construction Industry Research and Information Association, London. 73 p.

Yi, S. T.; Moon, Y. H.; Kim, J. K. 2005. Long-term strength prediction of concrete with curing temperature, *Cement and Concrete Research* 35(10): 1961–1969.