Prediction of concrete service-life

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This paper discusses development of accelerated tests and mathematical models for predicting the durability of concrete. Durability, service life, and degradation factors are defined and accelerated test methods are contrasted to conventional comparative methods. Factors and mechanisms of concrete degradation are reviewed, as are efforts to quantify these phenomena. Deterministic and stochastic models are discussed. Procedures for developing accelerated tests are presented and applied to a hypothetical example involving freeze-thaw damage. Advantages and disadvantages of accelerated testing and mathematical modeling are discussed in terms of the degradation mechanisms affecting concrete. Examples given of the modeling approach and service life prediction include the prediction of the strength and maturity of concrete, acid attack on cement, sulphate attack, and the effect of scaling and corrosion on load-bearing capacity of concrete.

1. INTRODUCTION

Experience and building codes incorporating factors of safety generally have proven adequate in the past for the assurance of concrete performance. They have helped the designer of concrete structures develop confidence that structures will function within their planned lifetime. But new materials, new construction techniques, and innovative designs have prompted a re-examination of traditional approaches. In addition, structures are often being used beyond their planned lifetime, for functions for which they were not originally intended, or are being subjected to harsher environments than anticipated. There is thus a need to develop comparative, in-service and accelerated tests to aid in the making of reliable predictions of service life.

This paper examines the basis for making service-life predictions based on accelerated testing of concrete and mathematical modeling of factors controlling concrete durability. Advantages and disadvantages of accelerated testing and mathematical modeling are discussed in terms of various typical degradation mechanisms to which concrete structures are subjected. Examples are given of the modeling approach and service life predictions.

1.1. Definition of durability, service life and degradation factors

Durability, service life and degradation factors are defined in ASTM E632-81, "Standard Practice for Developing Accelerated Tests to Aid Prediction of the Service Life of Building Components and Materials" [1], as:

durability, the capability of maintaining the serviceability of a product, component, assembly or construction over a specified time. Serviceability is viewed as the capability of these to perform the function(s) for which it is designed and constructed.

service life (*of building component or material*), the period of time after installation during which *all* properties exceed the minimum acceptable values when routinely maintained.

degradation factor, any of the group of external factors that adversely affect the performance characteristics of building components and materials, such as weathering, biological stress, incompatibility and use factors.

Both durability and service life incorporate the concept of design requirements being met or exceeded for a given time. They can be used as a basis for measuring the adequacy of durability analysis procedures. Frohnsdorff *et al.* [2] suggested the following procedure:

1. Make a quantitative estimate of the time to failure of a component or material when it is exposed to the expected operating conditions.

This estimate can be derived from either in-service-performance tests or from accelerated aging tests.

2. Obtain test results which correlate with in-service performance.

3. Identify the degradation mechanisms causing failures at high and low stress levels.

ASTM E632-81 defines accelerated aging tests as ones in which the degradation of building materials is intentionally accelerated over that expected in service. This is done in anticipation that the effects of in-service stress levels can be predicted from performance at accelerated stress levels. Proper identification of degradation mechanisms is important in extrapolating test results obtained from accelerated aging tests to normal in-service conditions.

Implicit in the testing method is the assumption that the in-service material will behave in the same manner as the accelerated sample, that is, that the degradative mechanisms will be the same in both cases. In another approach, non-accelerated tests can be used to measure small changes in properties when test samples are subjected to normal environmental conditions. Extrapolation of test results can then be made to predict service life. Implicit in this test procedure is the assumption that the extrapolation correctly mirrors the changes that would have occurred had it been possible or practical to conduct in-service tests.

Frohnsdorff *et al.* [2] present a list of the many generic factors that can affect the service life of building components and materials. Most existing tests do not incorporate all factors of importance, and those factors which are included may not be related quantitatively to in-service exposure conditions. Quantification of all the factors important for concrete durability would be impossible in terms of both time and money. Within these constraints only the most important factors can be investigated. This comment especially applies to accelerated testing where degradation mechanisms are often difficult to characterize.

Fagerlund [3] has recently suggested that service life is a better predictor of concrete structural performance than durability. He argues that service life is the more quantitative concept, so that structures should be designed based on anticipated service life. For each type of exposure, alternative concretes may give the same lifetime depending on such factors as type of cement, pozzolan, and admixture or plasticizers employed. Because of the complexity of such systems, prediction of the service life depends upon intimate knowledge of the degradation mechanisms. For example, while pozzolan usually improves concrete durability, addition of a certain pozzolan may lower solution pH, cause inadequate curing, increase water absorption and lower freeze-thaw resistance by changing the interlayer or spacing factor.

1.2. Limitations of conventional and accelerated tests

Both conventional durability tests and accelerated tests are subject to a number of limitations. Limitations often encountered in conventional tests include ([2], [3]):

1. Methods are rarely provided for correlating laboratory results with in-service results.

2. Provisions are usually not made for taking into account different applications.

3. In the laboratory, concrete specimens are usually tested in configurations substantially different from in-service configurations.

4. Recommendations are seldom made as to how the results of standard tests for different concretes should be compared with each other.

5. Quantitative estimates of time to in-service failure are seldom made, or possible, since most standard durability tests are comparative tests: where the durability of an unknown concrete is compared against the performance of a reference concrete, both exposed to identical conditions.

6. The factors affecting service life and the degradation mechanisms are complex, not well understood, and difficult to quantify.

7. Sensitive detection methods are needed since changes may be small under non-accelerating conditions.

8. Results may be extrapolated, often for exposure periods hundreds or thousands of times longer than normally feasible.

Many of the limitations listed are also of concern for accelerated testing. In addition, there are further limitations which can influence interpretation of test results. First, the time scale which maps the results from the accelerated tests to the in-service tests is unknown and probably nonlinear. Thus, even with identically known mechanisms, service life may not be reliably predicted. Second, acceleration can change the degradation mechanism, so that it is no longer the same as in the in-service test. For example, the use of accelerated temperatures is notorious for altering degradation mechanisms. Raising the temperature can cause a surface controlled process to become diffusion controlled. The morphology of the reaction products can also change. Lastly, synergistic effects between degradation mechanisms or non-linearities in the mechanisms can be present which can predict anomalous results under accelerated conditions. Correlation between accelerated aging and in-service tests is usually empirical at best and most likely introduces uncertainty in predicted service life.

Thus, the performance of a material in an accelerated test may not correlate with in-service performance, which may not be obvious. In spite of such limitations, it must be recognized that accelerated testing is often the only alternative. With this in mind, it is useful to examine conditions under which accelerated test results can be used.

2. PROCEDURES FOR DEVELOPING ACCELERATED TESTS

2.1. Recommended practices

ASTM E632-81 gives a recommended practice for the development of accelerated short-term tests for the prediction of service life. The practice consists of four main sequential parts (see *fig.* 1):

1. Problem Definition (steps 1-7).
2. Pre-Testing (step 8).
3. Testing (steps 9-12).
4. Interpretation and Reporting of Data (steps 13-16).

Part 1, problem definition, includes specifying in-use performance requirements and criteria (1) [1], characterizing the concrete (2), and identifying degradation indicators (3), factors (4) and mechanisms (5). Postulates are made of how degradation mechanisms of in-service performance can be induced by accelerated aging tests (6), and performance requirements are set for predictive service life tests (7).

Part 2, the pre-testing part of the recommended practice, entails the design and performance of preliminary accelerated aging tests (8). The purpose of these tests is to confirm the identification of degradation mechanisms and demonstrate the kinds and types of rapid failures caused by extreme levels of degradation stresses.

In the testing part (part 3), predictive service life tests are designed and performed based on the results of the pre-tests and relations found there between the severity of conditions and degradation rates of the concrete (9).

Long-term tests under service conditions are also designed and performed (10), and results are compared to the predictive service life tests (11). If the changes induced by predictive service life tests are similar to those observed in-service, then the final part of the recommended practice (part 4) is performed. If there are substantial differences, then all prior steps must be revised accordingly.

In Part 4, with the successful conclusion of the testing phases of the practice, mathematical models are developed describing degradation processes, to compare rates of change in predictive service life tests with those from in-service tests (13). As a result, performance criteria for predictive service life tests can be established (14). This leads to the prediction of service life under expected in-service conditions (15). The final step (16) involves reporting the data with regard given to presenting model assumptions and technology transfer.

[1] Numbers in parentheses refer to the step number in ASTM E632-81.

When degradation proceeds at a proportional rate by the same mechanism in both accelerated aging and long-term in-service tests, then it is possible [1] to define an acceleration factor K, such that:

$$K = \frac{R_{AT}}{R_{LT}},$$

where R_{AT} is the rate of change found from accelerated aging tests, and R_{LT} is the rate of change found from long-term in-service tests.

However, it is unlikely, given the complexity of concrete systems, with the many interrelated variables, that this approach would be reliable. Mathematical models must instead be developed to establish satisfactory relationships between rates of change from accelerated and in-service tests. These models should take into account the key variables and their interrelation-

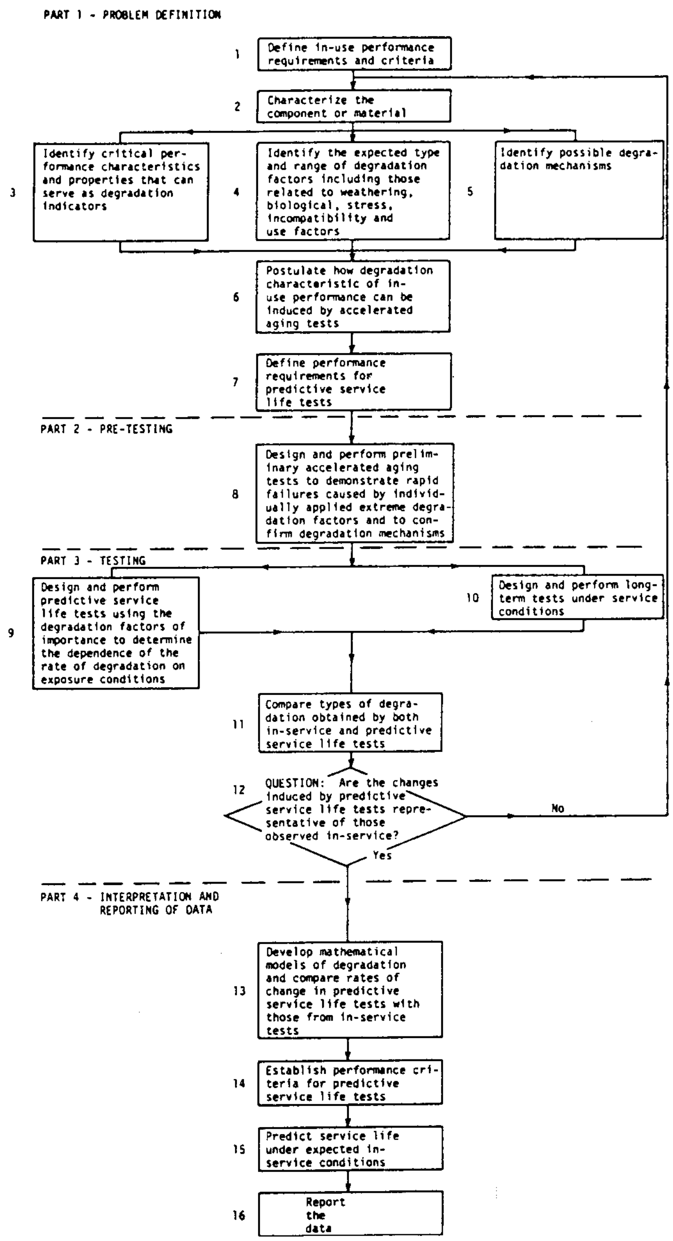

Fig. 1. — Recommended procedure given in ASTM E632 for developing predictive service life tests [1].

ships. They can either be mechanistic models based on known mechanisms, or based on data analysis using reliability theory. In either case, such models should be capable of using data from degradation tests to calculate rates of change.

2.2. Application to freeze-thaw damage

As an example, Frohnsdorff *et al.* [2] have considered the application of the ASTM recommended practice to the resistance of a concrete structure to freeze-thaw damage. Their example is based upon a hypothetical case developed by Plum *et al.* [4], which emphasizes modeling of degradation mechanisms.

Performance requirements and characteristics are related to compressive strength, which is used as an indicator of degradation. Freezing of water-saturated concrete is the only degradation factor considered, while the key degradation mechanism is fatigue associated with freeze-thaw cycling.

Accelerated aging tests are considered in which rapid freeze-thaw cycling occurs at high saturation levels (S). Loss of strength should occur by the same mechanism as that found in actual service and can be correlated with the number and intensity of the freeze-thaw cycles. For $S < 0.92$, tests were judged to be unnecessary.

They recommended that long-term tests be conducted under service conditions on similar concretes and the types of degradation compared to the accelerated tests to see if the mechanisms are sufficiently similar. Mathematical models could then be developed or modified to compare degradation rates in predictive tests to in-service tests. Strength was concluded to be the critical failure criterion. Development of a realistic model would allow predictions of service life under known conditions. The data and conclusions of such a study could then be appropriately reported, along with a discussion of the assumptions and statistical procedures employed.

3. THE MODELING APPROACH

Modeling has been described as "common sense made rigorous". Sometimes, though, common sense can be wrong. Since there is only one correct description, truth is elusive, and nature can obscure the truth. Properly designed and conducted experiments, coupled with observation and reflection, should be the arbitrator between competing interpretations or theories.

Mathematical models can be used to predict service life or other properties of concrete structures. The degradation processes occurring with either in-service or accelerated tests, if known, can be modeled. Mathematical models usually comprise a set of equations corresponding to the dominant or important physical and chemical phenomena or to their statistical description. The phenomenological or statistical description is referred to as the conceptual model.

The formulation of the model depends on the proper choice of such inputs as rate laws for degradation, constitutive or property relations, life distribution functions, and conservation equations for mass, heat and momentum transfer. With freeze-thaw damage, for example, models could be formulated based on simultaneous heat and mass transfer in a porous media. Although there are a number of ways to classify mathematical models, the most useful for our purposes is the distinction between deterministic and probabilistic models, since either can be used to predict service life. Considering strength of concrete as an example, deterministic models based on fracture mechanics would predict a single ultimate strength. Probabilistic models based on reliability theory would predict a range of ultimate strengths in terms of a life distribution function chosen considering the interaction of the concrete with its service environment.

Variability in results can be viewed mechanistically in terms of the competition between several time-dependent failure mechanisms. Such mechanisms are difficult to identify since failure initiates and propagates in one or more hostile microenvironments. Thus, two identical concrete samples exposed to the same macroenvironment, can fail at widely different times depending on local conditions. In such instances, probabilistic procedures are more useful than deterministic since they mirror the stochastic or unknown nature of the process. Martin [5] has recently reviewed the transformation functions that are commonly used in life testing analysis.

In an accelerated strength testing study, Moore and Taylor [6] developed a statistical method for predicting ultimate (28 day) strength from early time (48 hours) strength data. The method is conservative in that it predicts a lower incidence of failure for 28 day strength than is predicted by a standard regression analysis. This insures that "almost all" batches of concrete placed will meet the strength specifications.

3.1. Advantages and limitations of the modeling approach

It appears that modeling studies, if properly developed and conducted, have the capability of reducing the limitations of conventional and accelerated tests. Modeling itself, however, although a valuable technique, is not a panacea. It suffers from its own set of limitations, some of which are common to those found in testing. Table I summarizes the advantages and limitations of the modeling approach.

It would be ideal to be able to predict the properties of cement and concrete from first principles. That is, given the chemical and physical characteristics of the raw mix, the water to solid ratios and the environmental conditions, predict the strength and service life. If one had a set of equations that could reliably do this then it would be just as easy to work backwards and design a concrete system for a desired service life. Even knowing just the chances of failure would be preferable to the current state of affairs.

TABLE I

ADVANTAGES AND LIMITATIONS OF THE MODELING APPROACH

Advantages	Limitations
1. Can form basis for planning experimental work.	Mathematics does not replace observation.
2. Quantification, precise nature of mathematics.	Difficult to quantify some degradation factors and mechanisms; intimate knowledge needed.
3. Verification of conceptual model.	Alternative phenomenological descriptions may yield same results, can never "prove" a model.
4. Predictions of service life, incidence and mode of failure.	Degradation mechanisms may shift or change with time or environment.
5. Can extend to untested regions.	Extrapolations are risky.
6. Simulation models can predict continuous changes in concrete properties.	Accurate, non-empirical values of model parameters impossible to obtain from limited, inaccurate data.
7. Clear, unified, comprehensive view; sharpen focus.	Nature obscures the truth.
8. Can save money and time when used judiciously.	Improper use results in large wastes of time and money.

3.2. Applications of the modeling approach

As an example of the modeling approach, let us consider the prediction of the compressive strength of concrete. Factors affecting strength gain with time include cement and aggregate composition, particle size (fineness) and particle size distribution, water to solid ratios, additives and admixtures, methods of curing and mixing, and temperature. The environment and rate of loading can also influence strength development. At present the strength of concrete and cement cannot be predicted *a priori* since the precise influence and interaction of each of the factors affecting strength is not known. If such a prediction was possible, we would be on the road to predicting concrete properties from first principles. Nevertheless, some theoretical progress has been made, most notably in the areas of the prediction of the hydration rates of the constituents of portland cement, as well as the prediction of the strength of cement from a knowledge of its microstructure. The contribution of the aggregate to strength gain can be viewed as a problem in composite media. Since cement paste is often weaker than the aggregate, the contribution of the aggregate phase to overall strength development of concrete is not nearly as large as that of cement phase.

Phileo [7] has summarized the disadvantages of using 28-day strength as the sole measure of concrete durability. He views this approach as being too simplistic with no allowance for specimen history. Further, he shows data indicating that temperature tests (ASTM C683) are just as good as 28-day tests for predicting late-age (1 year) strength.

A number of investigators ([8], [9]) have attempted to correlate strength-gain empirically using the concept of maturity, as defined by equation (1):

$$M = \int (T - T_0)\, dt, \tag{1}$$

where M is the maturity, T is the temperature of the concrete or cement at time t, and T_0 is a datum or base temperature (often taken as $-12.2°C$). Although maturity has no theoretical basis, it does depend on the temperature history of the concrete, and, as such, represents an improved approach to predicting strength development.

The key idea in the maturity concept is that, for a given concrete, every maturity (M) gives rise to a single value of compressive strength. A number of strength-maturity relations have been proposed, all of them empirical, and most of them concrete specific ([8], [9]). A knowledge of the maturity and the strength-maturity relation should enable an investigator to more reliably predict strength. Equation (1) predicts that a concrete cured at high temperature for a short time should have the same maturity as one cured at low temperature for a suitably long time. Both would have equal maturities and nearly equal strengths. The maturity, then, represents a correlating parameter since it combines the separate influences of temperature and time. Since the maturity is an integral, the temperature does not need to be considered constant in equation (1). In some cases a controlled temperature program (T vs t) can be conducted to achieve a given maturity. For concrete members cured under field conditions, the temperature is uncontrolled, fluctuating with the weather and temperatures in adjacent structures, as well as internal heat generation caused by cement hydration. This leads to non-uniform temperature distributions within the concrete. The maturity will vary from point to point, generally being higher in the center of the member where temperatures are higher. The average maturity $\overline{M}$ can be defined by the volume (V) integral:

$$\overline{M} = \frac{1}{V} \int_0^V M\, dV. \tag{2}$$

For long cylindrical members of radius R this reduces to:

$$\overline{M} = \frac{2}{R^2} \int_0^R M\, r\, dr. \tag{3}$$

In general M will vary with both position and time, so that $\overline{M}$ will vary with time. From a knowledge of M and a strength-maturity relation, the average compressive strength of the member can be predicted at any time. Such an analysis does not consider the effect that local variations of temperature within concrete specimens will have on induced stresses.

In order to be able to predict strength-maturity relationships for cement and concrete, it is necessary to know both the strength and maturity as functions of time. At a fixed time, for any temperature history, the maturity will be completely determined from equation (1). The strength at the same time can be predicted theoretically from a suitable mathematical model which takes into account the rate controlling factors affecting strength. Unfortunately, at the present time there is no comprehensive model for strength development, either deterministic or probabilistic, which is based on first principles. If there were one, it would need to take into account hydration behavior. In this section we suggest one possible approach in developing such a model involving the concept of maturity, and discuss its limitations.

It is known that porosity is one of the most important microstructural parameters controlling strength. Pore volume of concrete and cement pastes are important factors in controlling the potential for the development of cracks and flaws which can propagate through the binding matrix. The properties of the flaw-free solid are also important since they will determine the ability of the solid to resist stress concentrations.

A simple but realistic mathematical model for strength has been derived by Bentur *et al.* [10], who were able to account for the strength gain with time caused by the formation of both physical and chemical forces between particles. As hydration proceeds, the porosity decreases and the binding energy between particles increases. They consider two different kinds of bonds: primary bonds and secondary (van der Waal's) bonds. For C_3S [1], C_2S and C_3S/C_2S pastes they estimated the contribution of secondary bonds from water surface area measurements and the contribution of primary bonds by silicate polymerization analyses. The contribution of $C-H$ bonds to the total bond energy was shown to be negligible in comparison to that of $C-S-H$ bonds. The data was correlated with the relation for predicting strength (f_c):

$$f_c = f_{cm} \exp(-bp), \tag{4}$$

where p is the porosity of the paste, b is a constant which depends on the pore size distribution and f_{cm} is the strength at zero porosity ($p=0$), or intrinsic strength.

It is assumed that the strength development of cement and concrete can be represented by equations similar to that of (4). This assumption is based on the fact that semi-logarithmic plots of strength versus porosity are often linear over a considerable range [11]. In addition, cement is composed principally of C_3S and C_2S, which together largely determines the observed strength. The strength of concrete will be determined principally by the strength of the weakest phase; usually the cement phase.

With the strength-porosity relations known, the strength-maturity relations can be predicted if porosity can be determined as a function of time. Porosity can be predicted at any time if the degree of cement hydration and density of the hydration products are known. For this purpose a recently developed mathematical model for C_3S hydration ([12], [13]) is applied. Hydration rates of cement and concrete are assumed to follow the same model. This assumption is made in the absence of any established comprehensive models for cement and concrete hydration [14], and in the presence of the fact that C_3S is the main constituent of Portland cement and is largely responsible for its strength development.

The C_3S model was based upon conceptual models for the observed hydration stages. The separate terms in the mathematical model corresponded to the phenomenological stages of the conceptual model. The conceptual model involved diffusion of chemical reactant species (water and ions) through ever-thickening layers of precipitated porous products to the interface between the C_3S core and the inner-most hydrate layer, dissolution of ions from the C_3S and their hydration near this interface, diffusion of soluble products back out through deposited hydrate layers, and precipitation of insoluble products as separate $C-S-H$ hydrate layers. A thin barrier layer is postulated as forming on the particle surface at very early times and then disappearing. It acts as a barrier to the diffusion of water into the particle and/or diffusion of solute ions away from the particle and is responsible for the induction period.

From a knowledge of the hydration stoichiometry, and the density of the phases, the solution porosity can be expressed as a function of the degree of hydration and the initial water to C_3S weight ratio. This amounts to making a volume balance on the hydrating mass and accounting for the separate volumes of the solid and liquid phases [13]. The porosity enters the model since the diffusivity depends on porosity [13]. Both growth of $Ca(OH)_2$ crystals in the bulk solution between particles, and the growth of hydrate layers around particles decrease the intergranular porosity, which in turn slows the diffusion process. The water to cement ratio (w/c) affects the analysis because more particles per unit volume will lead to lower values of the porosity.

As with most chemical reactions, the hydration of tricalcium silicate may be regarded as an activated process whose rate depends on temperature [15]. Higher temperatures accelerate both the reaction and the diffusion rate, although reaction rates are more sensitive to temperature than diffusion rates. It is reasonable to assume that beyond the acceleratory period, C_3S hydration is a diffusion-controlled process [13]. With diffusion control, the hydration rates probably would be much less temperature dependent, since, activation energies for diffusion usually are less than those

[1] Conventional cement chemistry nomenclature is used here: $C = CaO$, $S = SiO_2$, $H = H_2O$, $CH = Ca(OH)_2$.

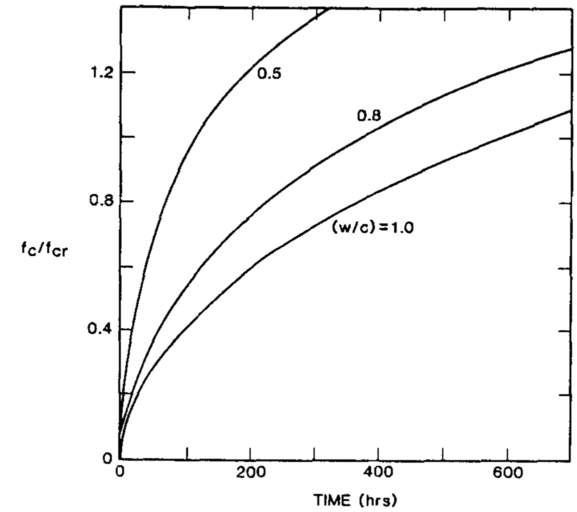

Fig. 2. – Strength ratio (f_c/f_{cr}) vs. time (hours). Effect of (w/c) : (w/c) = 0.5,0.8, 1.0.

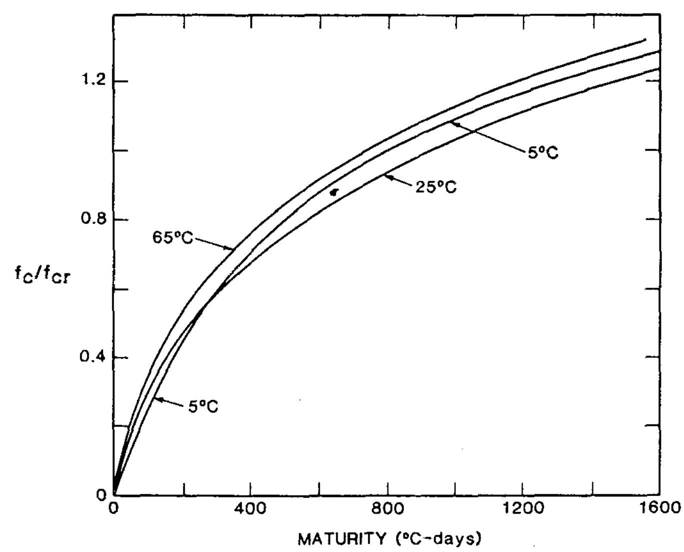

Fig. 3. – Strength ratio (f_c/f_{cr}) vs. maturity M (°C-days). Effect of temperature: $T = 5$, 25 and 65°C.

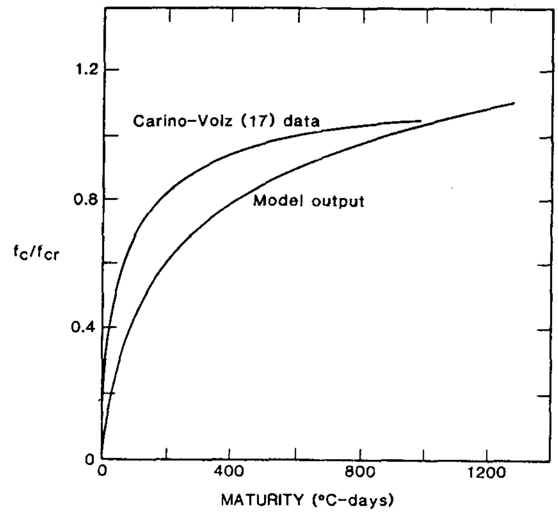

Fig. 4. – Strength ratio (f_c/f_{cr}) vs. maturity M (°C-days). Comparison with experimental data.

for reaction. In general, formation and disappearance of the barrier layer, as well as the gel layers, prevents the true rate constant from being measured in C_3S or portland cement hydration experiments, even when they are conducted in dilute solutions. Thus, the true kinetics of hydration is diffusion-disguised.

Using the C_3S hydration model to determine porosities ([12], [13]), and the strength-porosity relation of Bentur *et al.* [10], sample calculations can be made of strength and maturities.

For cement cured at 25°C, figure 2 shows model predictions of strength vs. time for three different water-solid ratios (w/c), with f_{cr} being the reference strength at 28 days. As expected, higher strengths are gained at lower values of (w/c) since lower values of (w/c) lead to lower values of porosity which cause higher strengths. For the model parameters employed, the compensating effect of lower hydration rates at lower water to solids ratios caused by lower diffusivities is not apparent in the results.

Figure 3 shows a plot of strength gain vs. maturity predicted by the model at temperatures of 5, 25 and 65°C. The curves do not coincide but they do predict strengths of similar magnitudes at equal maturities. The curve for the cement cured at the lowest temperature crosses that cured at 25°C to gain higher later maturity. Such an effect has been observed experimentally by Taplin [15], and is consistent with the interpretation by Verbeck and Helmuth [16] that densification of the hydrate layers at higher temperature lowers later strengths by increasing the solution porosity. No cross-over is predicted at the highest temperature, however.

The limitations of this approach are illustrated in figure 4, where strength-maturity data collected by Carino and Volz [17] for cement cured at room temperature are compared to model output. The agreement is not good, nor is it expected to be, given the large number of assumptions involved in the modeling.

This approach is useful but tentative. Most of the assumptions made to establish it have not been tested or verified. Specific limitations include:

1. Concrete is not cement, and cement is not C_3S/C_2S.

2. The maturity concept is empirical.

3. Strength is not just a function of porosity. Effects of other variables must be considered.

4. Most parameters in the hydration model are determined by curve-fitting hydration data rather than from experiments designed to individually measure each separate parameter (e. g., diffusivities through hydrate layers).

5. The conceptual model itself is just one of several competing theories, even for the hydration of C_3S.

6. Effects of temperature on physical and chemical processes is not sufficiently understood to allow accurate modeling.

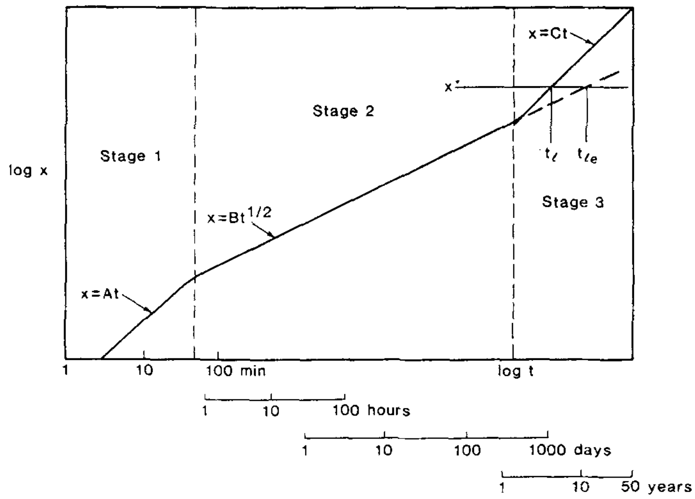

Fig. 5. — **Stages in acid attack of cement** [18], **depth of attack** (x) **vs. time** (t).

4. EXAMPLES OF SERVICE LIFE PREDICTION

4.1. Acid attack of cement paste

Romben [18] has examined the acid attack of cement pastes. As illustrated in figure 5, he suggests that there are three distinct stages of attack. During the first stage, which lasts several hours or days depending on the nature of the cement and the acid concentration, rate of attack is related to the surface area of the specimen. Since the mixing rate, acid concentration in the bulk solution and surface area are constants, the depth of the attack, x, varies directly with time. In the second stage, as a product hydroxide layer increases in thickness, the attack becomes diffusion controlled and x varies with the square root of the time.

Common sense might suggest that this is the final stage of attack, as would an analogy with cement hydration, where beyond the acceleratory period lower hydration rates suggest a diffusion-controlled process. But in acid attack, this is not the case. After several hundred days the rate of attack abruptly increases, and x once again becomes proportional to elapsed time, but with a different constant of proportionality. This third stage has been found to correspond physically to the dissolution of the hydroxide layer and the re-exposure of the cement surface to the bulk solution.

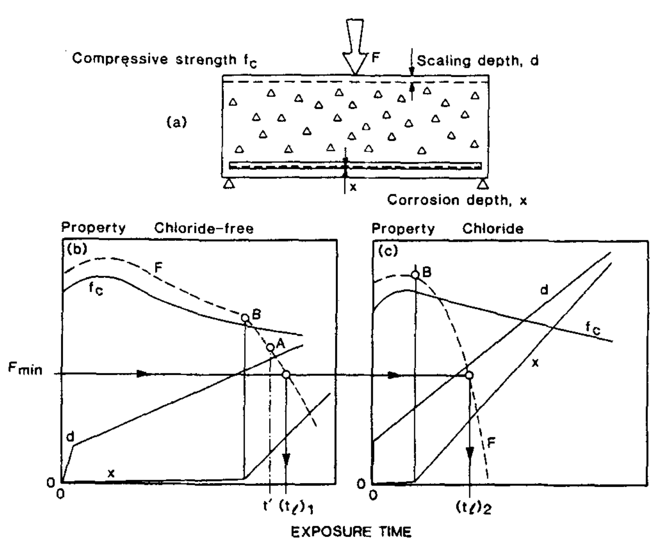

Fig. 6. — **Effect of corrosion and scaling on load-bearing capacity of concrete** [3]. *6 a* **Beam exposed to weathering,** *6 b* **Chloride-free environment,** *6 c* **Chloride-containing environment.**

Extrapolation of data in the second stage (without knowledge of the final stage) to predict the time where x would reach some critical unsatisfactory value x^*, would predict failure at time t_{le} which is considerably longer than is actually the case (at time t_l). Because the time scale in figure 5 is logarithmic, what appears to be a small difference in actual and predicted failure times can represent many years of unattainable service life. Fagerlund [3] points out the importance of making sure that non-accelerated tests are carried out for a sufficiently long time. The problem is, this can only be determined after the fact.

Whether accelerated tests would clarify the analysis is debatable. Accelerated attack would involve increased acid concentrations, higher temperature or both. Dissolution of the hydroxide layer would be accelerated and in some cases it might not even form, so that the second stage would be missing. Higher temperatures would also increase diffusion rates, but the magnitude of this increase compared to dissolution rates is not known. If the non-accelerated test results can be deceptive, it is doubtful that accelerated test results can give more definitive results.

4.2. Sulfate attack of concrete

The importance of properly identifying degradation mechanisms, and the range of degradation factors in both developing accelerated durability tests and in modeling deterioration processes can be demonstrated by analyzing the sulfate attack of concrete. At least three major deleterious sulfate reactions can take place depending on the exposure conditions ([19], [20]).

The first reaction considered involves the replacement of $Ca(OH)_2$ in concrete by $CaSO_4.2H_2O$ (gypsum):

$$Ca(OH)_2 + Na_2SO_4.10\,H_2O$$
$$\rightarrow CaSO_4.2\,H_2O + 2\,NaOH + 8\,H_2O.$$

Formation of gypsum can lead to the deterioration of concrete by two processes. In one process, because gypsum occupies more volume than calcium hydroxide, expansive stresses are produced. In another, gypsum is leached, leaving a porous concrete with a higher permeability. In flowing water, calcium hydroxide may be completely converted to gypsum, while in stagnant water, equilibrium will be attained, and only a portion of the calcium hydroxile will be converted.

Another expansive process involves the reaction of sulfate ions with calcium aluminate hydrate to form the calcium sulfoaluminate product, ettringite ($3\,CaO.Al_2O_3.3\,CaSO_4.31\,H_2O$):

$$2(3\,CaO.Al_2O_3.12\,H_2O)+3(Na_2SO_4.10\,H_2O)\rightarrow$$
$$\rightarrow 3\,CaO.Al_2O_3.3\,CaSO_4.31\,H_2O+2\,Al(OH)_3$$
$$+6\,NaOH+17\,H_2O.$$

Ettringite has a considerably larger volume than the reactants. At low concentrations of sulfate ions, ettringite decomposes to a low sulfate form $3\,CaO.Al_2O_3.CaSO_4.12\,H_2O$. The amount of ettringite accumulated and stress induced, therefore, depend on the availability of sulfate ions.

Similar reactions involving magnesium sulfate occur when concrete is exposed to sea water. In addition, magnesium sulfate can attack the calcium silicate hydrate gel, the major strength producing reaction product of cement hydration:

$$3\,CaO.2\,SiO_2(aq)+MgSO_4.7\,H_2O$$
$$\rightarrow CaSO_4.2\,H_2O+Mg(OH)_2+SiO_2(aq).$$

This reaction has particularly severe effects. Because of the very low solubility of magnesium hydroxide, the equilibrium is almost completely to the right.

Thus, in developing accelerated tests for modeling sulfate attack, two reactions need to be considered if sodium sulfate is present, or three reactions if magnesium sulfate is also encountered. If both are present, then their relative concentrations, as well as the kinetics of the respective reactions, must be known. Further complicating the simulation of sulfate attack is the extent of the hydration of the portland cement, which controls the strength of the concrete, the amount of calcium hydroxide, calcium aluminate hydrate, and calcium silicate gel available for reaction, and the permeability of a given concrete. Other factors also need to be considered, e. g., is the water flowing or is it stagnant; and, if it is stagnant, are the consumed sulfate ions being replenished? Further, the possibility of acid attack involving sulfuric acid, or the effects of salt crystallization when concrete is exposed to periodic conditions of wet and dry are not included in the above treatment.

To realistically model sulfate attack the reaction mechanisms need to be well understood. In addition, information is required on the diffusion constants for the soluble reactants and products, the kinetics of the reactions, and their relative deleterious effects.

4.3. Service life of a supported beam

Fagerlund [3] considers a hypothetical example in which a supported beam made of reinforced concrete, and having compressive strength f_c, is subjected to a centered load F. Compressive strength is gained by hydration and lost by fatigue loading. The environment is considered to be moist, and to contain either chloride or to be chloride-free. The situation is shown in figure 6.

Alkali-aggregate damage is presumed to be absent, with frost damage being assumed to occur only in the (top) concrete cover, which results in scaling with an effective depth d. The corrosion depth of the reinforcement is denoted by x. Increases in x and d with time lower the loadbearing capacity F.

In chloride-free but moist environments (*fig.* 6 *b*), corrosion does not begin until the cover has been completely carbonated. This is a fairly slow process. Chloride, on the other hand (*fig.* 6 c), can diffuse through concrete rather rapidly, and is considered to cause corrosion to initiate much earlier and to proceed at a faster rate. Oxygen transfer, moisture content and concrete permeability also influence corrosion rates. These in turn are controlled by (w/c) ratios and the type of cement.

Scaling depends on both moisture content and salt content. Scaling can be prevented by using an air-entrained concrete cover with low (w/c). Once scaling begins it is considered to proceed at a linear rate, a rate which is higher if chloride is present. Scaling also reduces the effective depth of the beam, and thus its load bearing capacity F. Standard structural calculations can give the changes in F as a function of the dimensions x and d, at least to a first approximation.

The service life t_l will be that time at which F is lower than F_{min}, the minimum acceptable load-bearing capacity. If the concrete cover spalls off first at time t' (*see* point A in *fig.* 6 *b*), then catastrophic collapse will occur earlier.

Fagerlund [3] draws four general conclusions from this example: (1) Service life depends critically on the environment. Equal service lifes are obtained in both chloride and chloride-free environments only if a superior concrete is used when chloride is present. The (w/c) ratio must be decreased to prevent corrosion and the air content raised to prevent scaling. The (w/c) ratio and air content which should be selected are those which lead to equal service lifes; $(t_l)_1=(t_l)_2$ in figure 6. (2) Degradation mechanisms can change abruptly. In this example the controlling mechanism changes from frost attack to corrosion at point B in figure 6. (3) Reliable service life predictions cannot be made unless degradation mechanisms are known and environmental factors quantified. Reliability analyses are viewed as being only of general applicability since they do not apply to specific structures, but only to a large number of structures subject to standard environments. They are of minimal use in designing durable concrete for new structures in new environments. (4) Service life predictions are possible only if the functional requirements are precisely specified and quantified. For this example, since the functional requirement is load-bearing capacity, service life can be expressed in terms of concrete strength, and scaling and corrosion depths. If the functional requirement was deflection instead, Fagerlund concluded that time dependence of thermal expansion, shrinkage and creep would become important.

5. CONCLUSIONS

The prediction of the service life of concrete structures is largely based on experience and this approach generally has been satisfactory. The use of new and innovative materials, construction practices, and designs have prompted a reexamination of traditional approaches. In addition, often structures are being used beyond their planned lifetime, or for functions for which they were not originally intended, or are being subjected to harsher conditions than anticipated. There is a need, therefore, to develop an improved approach for predicting the durability of concrete. Accelerated testing which simulates the exposure conditions to which a structure is subjected, coupled with the development of mathematical models, is a promising approach for making improved predictions. The constraints involved in applying such analytical approaches need to be understood; nevertheless, these approaches have the potential to provide the framework for significant improvements in the durability of concrete.

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RÉSUMÉ

Prévision de la durée de service du béton. — *On examine la mise au point d'essais accélérés et de modèles mathématiques pour prédire la durabilité du béton. On définit la durabilité, la durée de service et les facteurs de détérioration, et on met en comparaison les méthodes d'essais accélérés et les méthodes conventionnelles. On passe en revue les facteurs et les mécanismes de détérioration du béton et les efforts entrepris pour mesurer ces phénomènes.*

On examine des modèles déterministes et stochastiques. On présente des procédés de mise au point d'essais accélérés et on les applique à un exemple hypothétique de détérioration au gel/dégel. On envisage les avantages et les inconvénients de l'essai accéléré et des modèles mathématiques par rapport aux mécanismes de détérioration affectant le béton. Les exemples d'études sur modèles et de prévision de la durée de service comprennent la prévision de la résistance et de la maturité du béton, de l'attaque du ciment par les acides, par les sulfates, et de l'effet de l'écaillage et de la corrosion sur la capacité portante du béton.