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Some issues related to service life of concrete structures

B. Bhattacharjee

Current specification of concrete relies mostly on 28 days strength and mould-ability that is quantified through some empirical indicators such as slump, Vebe time etc. The last mentioned indicators are determined through respective engineering tests. The long term durability however is handled only through prescriptive measures, say, by specifying max w/c ratio, minimum cement content and minimum recommended grade of concrete. Ideally, durability shall be quantified in terms of time during which the concrete element is able to perform its designated function without exhibiting signs of any unacceptable deterioration in a given exposure condition. Defining life of structure is somewhat ambiguous and confusion persists, as unlike living beings, clear-cut demarcation event of death, that separates itself from the life, does not exist for structures. To avoid confusion in this article, life related concepts and definitions of certain terminologies are presented in the beginning, complexities involved in service life modelling are illustrated and inadequacy in current service life modelling is described. Hence, a discussion on durability performance specification of concrete in Indian context is presented.

Life related concepts for concrete structure

Thermodynamically only naturally existing materials can be stable over a long period of time as they would be in a state of minimum or nearly minimum potential. Man made materials those are produced at the expense of energy, would tend to dissipate this energy and undergo chemical changes in reaction. Concrete is produced with a considerable expense of energy and thus under conducive conditions would undergo chemical changes. Quite often, during its service condition, the concrete gets exposed to the aggressive environment that causes deterioration. Deterioration is the process of becoming impaired in quality and value. A chemical reaction leading to changes having no effect on concrete performance may not be considered as deterioration. Degradation process in concrete may take place due to sulphate attack, frost action, action of acids, alkali-aggregate reaction and in steel reinforced concrete, the degradation may also take place by carbonation and chloride ingress leading to corrosion of rebar. As a result of degradation, deterioration of concrete takes place with time. Degradation by definition is gradual decrease in performance with time. With reference to concrete structure, performance can be understood as the behaviour related to the use of the structure. Hence, the performance can be related to bearing capacity, stability, and safety in use, tightness against ingress, hygro-thermal properties, visual appearance etc. For concrete, these behaviors are dynamic and changes with time. Therefore performance is a function of time and is related to durability and degradation of concrete. Durability is a property expressing the ability to maintain the required performance. Therefore, degradation is the opposite of performance and often serves as another way to treat performance problem. Hence it is desired that concrete should not deteriorate in the envisaged service environment at a rapid rate and must continue to perform satisfactorily for a desired service life. Service life can be understood as the period after inception of the structure, during which all the essential properties of the structure meet or exceed the minimum acceptable value[1,2]. The long-term performance of concrete is related to its durability related properties. The concrete as a material shall therefore be so designed, that the structure where it is to be used should posses the designed service life with minimum possible maintenance requirements. The economic objective of the material design can be minimization of life cycle cost that includes the maintenance cost as well. It may be noted however that although manmade materials tend to deteriorate, the tendency does not necessarily mean deterioration would actually take place as; the same is also governed by the kinetics or the rate of the process. For example; significant deterioration would require infinitely long time for an infinitesimally small deterioration rate. While for faster rate of deterioration, the degradation of the structure would lead to rapid performance failure.

The answer to the question, how long a structure is going to last or what is the life, is not straight forward as end of life can be defined in several ways depending upon context, for example; physical life, functional life, economic life and service life etc. Again there can be intended design life that is the life envisaged in the, design stage. It is the life during which structure is expected or intended to be in service. Intended design life is used in the context of design load calculation and it also represent the planning horizon at the design stage. Physical life is the period of actual survival of structure. Most of the time, the physical life can be prolonged desirably, with adequate maintenance, hence infinite for all practical purposes. Functional life relates to change of function of the structure, e.g., a cinema theatre converted in to a shopping complex with some alteration in space. Occupancy class would change hence imposed load and importance factor etc may change accordingly. Economic life relates to increased cost of its maintenance rendering the structure economically unworthy of future continuance vis-à-vis an alternative option of demolition and possible reconstruction. Physical life of a structure can be very long before it collapses if properly designed and constructed, compared to economic life and functional life. The period during which the structure would remain in service is the minimum of physical and economic life. Service life can be appropriately related to serviceability limit through the concept of limit state design. Design procedure of RCC structure is carried out by sizing individual member elements and their reinforcement contents. Similarly, pres-stressed concrete members are designed by sizing the member and their corresponding level of prestress. Serviceability limits also referrers to individual member, hence service life of RCC/PSC shall refer to elements. The time when deterioration leads to degradation of an element to an unacceptable serviceability limit, is the service life of that element. This time can be considered from the time of inception of the structural element. Overloading of structure causes damages over the short period loading, e.g., loading due to extreme earthquake and cyclone can cause almost instantaneous damage. As per prevailing design philosophy, structure is not expected to collapse even in case of probable extreme events mentioned above, therefore the limit state of collapse is considered in the corresponding structural designs. Further such event is expected to occur not more than once during intended design life. Design against deterioration on the other hand is not related to collapse as the process of deterioration is slow, can be detected early in most cases and the impending danger of collapse can be avoided by appropriate and timely repair of element. Service life thus is related to repair cycle of the element and maintenance of the structure. In some cases the whole structure may require rehabilitation. Service life therefore can also be considered from the time of last repair. To increase the reliability against loading, redundancy in terms of excess capacity is always ensured in structure, so, even if one member becomes incapable of carrying the load other members share it by redistribution. Thus the capacity to redistribute the load provide additional cushion against failure to collapse due to deterioration. Service life of concrete elements therefore is related to maintenance cost rather than collapses, as deterioration rarely leads to sudden collapse of the structure. Maintenance of concrete structures is a necessity; economic and appropriate maintenance policies and repair strategies for concrete structures can be formulated through service life[2-5]. The exposure environment, concrete material and quality etc., control the rate of degradation and hence the service life of concrete structural member. Actual time of attainment of serviceability limit relates to cover depth and varies from member element to member element as; neither the exposure condition of all elements are same nor their resistance against deterioration is identical. The uncertainties of exposure environment and material shall be accounted for in service life design of structure.

Service life assessment

The service-ability limit with respect to a specific deterioration needs to be defined for estimation of service life. For example with respect to corrosion of rebar, appearance of first visible crack is often defined as the service-ability limit. Consequently, the service life is defined as the time from inception to the time of appearance of first visible crack. Similarly, service-ability limit and hence time for reaching the limit i.e., service life in the context of all other deterioration mechanism can be defined. Service life can be estimated when rate of deterioration is known. The time required for attainment of service-ability limit can be assessed from the knowledge of service-ability limit and deterioration rate.

Deterioration rates again depend on reaction kinetics and available concentrations of reactants in the concrete. At least one of the reactants must be present in the concrete and at least another reactant must be entering in to the concrete by ingress. Most of the deterioration processes in concrete take place in presence of moisture and due to ingress of aggressive chemical agents those cause deterioration. The aggressive agent again ingress in to concrete usually in solution phases with water or are fluid themselves. Concrete inherently is a capillary porous material by nature. Thus its permeability and diffusion properties are important from its durability performance point of view. Modelling of deterioration rates would involve considerations of ingress mechanisms such as permeation, diffusion, capillary suction, reaction kinetics etc. First and foremost in this context is modelling for moisture profile. Quite often reaction occurs only in a conducive moisture condition e.g., carbonation occurs most rapidly at relative humidity ranging from 55-65%, and, do not occur either in fully saturated or completely dry concrete. Similarly rebar corrosion can occur only in a partially saturated concrete. Therefore modelling wetting and drying of concrete is most important in the context of service-life assessment of concrete. In case of exposed concrete wind driven rain is driving force for moisture ingress and modelling for moisture profile in concrete in main three tropical climatic zones of India would be quite relevant in this context. Second aspect of service life models would be to model ingress of aggressive agent causing deterioration. The process involved may be concentration driven diffusion. For example, in case of carbonation, atmospheric CO_2 may diffuse in to concrete under concentration gradient. Ionic diffusion of chloride ion due to concentration gradient can also be used to model (free) water soluble chloride in saturated concrete. However, it is important to recognize physical and chemical binding of chloride in concrete; hence Fick's diffusion equation needs appropriate modification. Modelling of corrosion process however, is extremely complex involving diffusion of oxygen, Fe^{++} ion, moisture etc in addition to mass and charge balances.[6] Thus mass balance and stoichiometry of reacting species is the third aspect in modelling deterioration process of concrete. The last aspect is the estimation of stresses and resulting crack propagation due to formation of expansive product wherever applicable. Thus service life assessment of concrete requires modelling of complex physical and chemical process involving several compounds. At the moment, modelling of most of the deterioration processes is not well understood and hence most of the available deterioration models are inaccurate. The inadequacy of deterioration models are discussed further in the next section.

A second aspect of complexity in service life assessment is the interaction of loading with deterioration environment, while static or quasi-static imposed load may not significantly affect the deterioration, fatigue loads with stress reversals on the other hand may enhance the deterioration process through crack initiation.[2-5] Shrinkage, thermal and other intrinsic cracks similarly would also enhance the progress of deterioration.

A third aspect in this context is the difficulty of calibration and validation of deterioration models against real life behaviour. The manifestation of deterioration is visible only after very long exposures and any validated models are not available. Determination of coefficient of the model and material properties poses similar problem due to lack of understanding of the phenomena. For example determination of chloride diffusion coefficient from a migration test under electric field is meaningless, as, driving forces under natural concentration driven diffusion and that under imposed electric field are different. In the former Chloride and cation may move together in the same direction where as in the later they would invariably move along opposite directions.

Current state of degradation models

An elaborate review on degradation phenomena with reference to durability of concrete is presented by Glasser et.el.[7] In their article the authors first look in to general transport mechanisms followed by chloride ingress and corrosion, carbonation, decalcification and sulfate attack. From their review, authors conclude that results of accelerated test proposed so far cannot be extrapolated to real life and progress towards analytical modelling is uneven. The authors further expect a transition of durability research from empirical state to quantitative basis in future. In the next sub-section two deterioration phenomena namely; chloride ingress and carbonation are considered to illustrate the current state of deterioration models. These two phenomena are not deteriorations of concrete per se, however depassivation of rebar resulting from these processes initiate corrosion leading to serviceability failure of RCC elements. The rebar corrosion being the major durability concern in RCC structures these phenomena had attracted maximum attention in practice.

Carbonation

The carbonation model essentially relate depth of penetration of carbonation front with time. Most commonly used and simplest form of semi-empirical degradation model for carbonation is $d = k_c t^{1/2}$; where d is the depth of penetration of carbonation front, t is the time and k_c is a coefficient depend upon concrete material as well as environment and can be determined empirically.[1] Service life of concrete with respect to carbonation is defined as the time when depth of penetration of carbonation front becomes equals to cover depth of rebar. Thus time t at which this depth d becomes same as cover depth can be easily estimated from above formula. More elaborate models are based on diffusion equation and can be written as:[7]

$$\frac{\partial(\phi - w)[CO_2]}{\partial t} - \frac{\delta}{\delta x}\left((\phi - w)D_c \frac{\delta}{\delta x}[CO_2]\right) - f_c = 0 \qquad (1)$$

where ϕ is the porosity, w is the volumetric water content, $[CO_2]$ is the carbon dioxide concentration, D_c is the diffusion coefficient and f_c is a sink term dealing with reaction of carbon dioxide with alkaline materials in concrete. Two issues were highlighted by the authors namely; phenolphthalein indicator based measurement tends to under estimate the carbonation depth. Secondly most of the models ignore the pH drop by overlooking OH^- concentration. Actually pH drop is responsible for depassivation. However, some experimental evidences tends to justify $d = k_c t^{1/2}$ relationship although realistic models dealing with depassivation are non-existance.[7,8]

Chloride Ingress

Chloride ingress again is not a deterioration per se, but in RCC, chloride concentration above a threshold level can initiate rebar corrosion. Service life with respect to chloride ingress is considered as the time required for chloride concentration at the rebar depth to reach the threshold level. The mechanisms involved in chloride ingress can be ionic transport and capillary suction in unsaturated concrete. The driving force for ionic transport may be concentration gradient, chemical activity, potential difference due to ionic concentration etc.[9]

$$q_{cl} = D_{cl}\left(\frac{dC_{cl}}{dx} + C_{cl}\frac{d\ln a_{cl}}{dx} + \frac{zF}{RT}C_{cl}\frac{dV}{dx}\right) + C_{cl}q_w \qquad (2)$$

In equation 2, q_{cl}" is flux, "D_{cl}" is chloride diffusion coefficient, "w" stands for moisture, "C_{cl}" is water soluble chloride concentration, "a" is activity and "V" is potential due to interaction of various ionic species. q_w is solution flux and can be obtained from extended Darcy's law for unsaturated porous materials.[10] Considering mass balance:

$$\gamma\frac{\delta C_{cl}}{\delta t} = \frac{\delta q_{cl}}{\delta x} \qquad (3)$$

γ is a chloride binding term, and combining equations 2 and 3 one would get the equation for chloride ingress in unsaturated concrete. All the material coefficients for use in equation 2 and 3 are not available. Therefore use of these equations at the current state is precluded. Quite often people use the following equation neglecting electrical coupling, chemical activity and chloride binding.[7]

$$\frac{\delta C_{Cl}}{\delta t} = D_{Cl}(app)\frac{\delta^2 C_{Cl}}{\delta x^2} \qquad (4)$$

$D_{cl}(app)$ stands for apparent chloride diffusion coefficient. Equation 4 is only valid for saturated concrete and obviously it does not represent the real situation in any manner. Yet this equation is often used in service life prediction of real structure with following set of specific initial and boundary conditions for which closed form solution is available in heat transfer.[11]

For a infinitely thick wall; for $t< 0$; $C_{cl}(x,t)=C_0$; and $t \geq 0$; $C_{cl}(0,t)= C_s$; $C_{cl}(\propto,t)=C_0$; the solution is:[11]

$$C_{cl}(x,t) = C_s + (C_0 - C_s)\,erf\left(\frac{x}{2\sqrt{D_{cl}(app)t}}\right) \qquad (5)$$

where,

$$erf(y) = \frac{2}{\sqrt{\pi}}\int_0^y e^{-u^2}du$$

The boundary conditions mentioned above is rarely encountered in real life situations and use of the above model for estimation of cover depth for a given service life of concrete element can provide only intellectual satisfaction of using a model and would be totally unreliable. The cover depth proposed on the basis of experience and consensus of many experts is far more reliable. For example, using this model for determining safe cover depth against chloride ingress for 125 years for a tunnel lining at 20m below ground level, where acid soluble chloride concentration in soil was known, is totally erroneous. The chloride flux deals with free water soluble chloride not acid soluble ones, further the concrete is unsaturated, hence neglecting solution flux is incorrect. It may be noted that solution flux contributes much more than the diffusion flux. Chloride concentration C_s at the surface is not same as that in the soil and would depend upon surface current. Temporal constancy of C_s over 125 years is also questionable. So far as the inappropriateness of use of the equation 5 to a specific case. In general following are the additional problems associated with modelling of chloride ingress in general, more so when modelled through equations 4 and 5.

First of all, there is complete uncertainty regarding threshold chloride that is the backbone of chloride ingress modelling. The threshold chloride concentration depend upon cement type, steel type etc, hence no single value is available.[12,13] Secondly, there is no reliable way of measurement of diffusion coefficient as it is not possible to isolate the phenomena involved, besides, diffusion coefficient cannot be measured reliably over a short period of time.

Degradation models for other deterioration processes are still in the development stage and as such service life assessment concept is yet to mature sufficiently before it becomes possible for its use in real life practical design situation.

Service life design philosophy

Reliability based design concepts are well accepted principle and one can describe the failure event in terms of two variables i.e., load variable S and resistance variable R. The failure than can be defined as: $\{failure\}=\{R<S\}$; In the case of service life design the resistance is a function of time and even load can be similar. Failure probability is thus defined as $P_f(t)=P\{R(\tau)<S(\tau)\}$, for $\tau \leq t$. R and S are considered to be stochastic quantities with time dependent or constant density distributions[1]. There can be two principles namely; (i) performance principle and (ii) service life principle. In the performance principle R and S needs to be related and then evaluated according to the reliability concepts mentioned above. In the service life principle: service life t_L needs to be estimated and compared with target service life t_g using same reliability concept above. There can be, deterministic, stochastic and life time safety factor approaches. In the deterministic approach performance principle leads to the condition that $R(t_g)-S(t_g)>0$; while service life principle requires that the condition $t_L-t_g>0$. In the stochastic approach the requirement corresponding to performance based principle can be expressed as: $P\{failure\}_{tg}=P\{(R-S)<0\}_{tg}<P_{fmax}$; where P_{fmax} is the maximum allowable failure probability. The requirement in case of service principle can be expressed as $P\{failure\}_{tg}= P\{(t_L-t_g)<0\}<P_{fmax}$; In both the cases distribution of R, S and those of t_L need to be known either from available data or reliable simulation. However, neither seems to be feasible at the current state of research. In the life time safety factor method the design service life is obtained multiplying target service life by a factor of safety as $t_d=\gamma_t t_g$; where γ_t is life time safety factor. correspondingly the design formulae becomes $R(t_d)-S(t_d))\geq 0$; and $t_L-t_d\geq 0$ respectively for performance and service life principles respectively.

The design concepts discussed above have been applied in many fields of mechanical and structural design, however their applicability in case of service life design of concrete element is limited by limitations of the basic degradation models mentioned earlier. This is because the degradation models are the basic tool for estimation of either R or $t_L$etc.

Residual service Life

At many instances during the service period of structure, one may be interested to estimate the remaining or residual service life of concrete elements as well as for the structure as a whole. Residual service life assessment requires obtaining first hand information regarding the current condition of the structure through a thorough condition survey. Such condition survey involves non-destructive and semi-destructive tests to obtain the strength and other properties of the concrete. Thorough visual survey and document surveys are essential in condition survey for getting a clear picture about the type of distresses existing in the structure, if any. This type of investigation of existing condition is intended to determine the state of the health of the structure, establish a diagnosis and to arrive at a prognosis. Through the prognosis one can estimate the expected residual service life using some of the degradation models mentioned earlier.[14,15] For an existing structure condition of elements can be expressed in terms of condition index through inspection and survey and condition index of all elements can be combined to obtain overall condition index for the structure as a whole.[16-17] Such condition index indicate the repair priority. Through prognosis change of condition index with time can be estimated. However, prognosis is limited by limitations of service life assessment, although this may be easier than service life assessment of new structure as one can make more realistic assumption regarding exposure environment.

Material properties and state of degradation can be known through condition assessment. When condition assessment is done at regular interval through a planned inspection scheme, performance can be related to time either empirically or through model or using both concepts together. One may take advantage of Markov's theory to predict the future state in such situations.[1,14]

When expected service life of structure as whole is desired, the safety against load also needs to be considered. For the available strength the safe wind pressure that can be withstood can be estimated and corresponding design wind speed, basic wind speed etc and the return period of the later can be determined using appropriate distribution.[18] The expected service life would have a bearing on the above return period. Similarly, for the given materials and structural system determined and identified through appropriate condition assessment, the degree of tolerable ground motion (acceleration, velocity and displacement etc) and the resulting tolerable forces, moments and overall tolerable hazards for the structure can be estimated, and hence corresponding size of the earthquake in terms intensity and magnitude can also be estimated. Through probabilistic seismic hazard analysis the period of time during which this hazard would not likely to occur more than once can be estimated, this period will also limit the expected residual service life of the structure[19].

Closing remarks and discussions

Discussions presented makes it clear that adoption of service life design in practice for practical construction is premature at this stage because of (i) conceptual limitations in the degradation models, lack of understanding of the phenomena involved, (ii) lack of real time validation or calibration against performance in actual structures and lastly, (iii) difficulty in determination of material coefficients and other parameters in the model. Further, assessment of service life through accelerated test although had been attempted in past needs further research prior to their adoption in practice.[7,20]

The present exposure conditions given in IS 456:2000 needs a relook. Exposure classification can be more elaborate and India specific. For example the temperature, the relative humidity and rainfall pattern govern the moisture conditions and reaction rates in exposed structures. Such conditions would depend upon climatic zones, namely, hot-dry climate as in Jodhpur, warm humid climate as in Kolkata and composite monsoon climate of Delhi. A concrete exposed in warm humid climate is likely to have a conducive moist condition for deterioration for a longer period of time with its longer duration of rainy season annually compared to that in hot-dry desert climate where rainy season is for short duration.[21] Geographical location and geological conditions again are the other considerations those shall go in to exposure classification. Exposure in coastal areas would be different than locations away from cost. There again, the surface facing sea may be more conducive for deterioration compared to opposite face. Analysis of current deterioration pattern may not always provide reliable answer to above requirements. Quite often concrete structures in north India had been constructed with chloride ridden ground water resulting in early corrosion distress.[22] Thus analysis of current distress would indicate most of the northern India being rebar corrosion prone due to chloride, there by leading to erroneous inferences. Loading conditions such as static or fatigue loading also shall be taken in to consideration.

The present prescriptive practice recommended in IS 456:2000 also needs a relook. While, options of cement/cementitious combinations and other ingredients of concrete, limits of water cementitious ratio etc., may be suggested, inclusion of some measureable performance parameter in specification of concrete may provide a reliable way of ensuring adequate durability of concrete at present . A number of qualitative tests such as water absorption, Initial Surface Absorption Test (ISAT), Figg's air permeability, etc are available for relative evaluation of permeation quality of concrete. Limits of such permeation property may be proposed. [23,24] Determination of properties involved in models would require consideration of micro structure and their relationship with permeation and diffusion properties. Attempts have been made to relate these properties to micro parameters such as pore size parameters and mix factors as well. [25-30] The importance of the structure may also be considered while proposing the prescriptive recommendations on material options, cover depth etc., as intended maintenance free service life may vary from structure to structure.

Lastly, with sustainability concern the specification of concrete may include specification with regard to sustainability also. This is because sustainability encompasses durability as well. A sustainability index is presented recently that incorporates implications of repair cycles in the index itself together with other concerns.[31]

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Dr. Bishwajit Bhattacharjee is Professor at the Department of Civil Engineering, Indian Institute of Technology Delhi-India. After obtaining B.Tech (Hons.) degree from IIT Kharagpur, he worked for M/s Gammon India Limited for a short period of two years. Subsequently he obtained M.Tech. and PhD degrees from IIT Delhi. His areas of active research interest includes corrosion of rebar in concrete, high-performance concrete, microstructure modelling of concrete, chloride ingress, service life prediction and life cycle costing of concrete structures besides condition evaluation and health monitoring of structures. Minor area of research and teaching interests are Building Physics and Building Engineering, Construction Technology and Construction management. He is Fellow of Indian Association of Structural Engineers. He is Life member of 1) Indian Concrete Institute, 2) Indian Society for Construction Materials and Structure. 3) Indian Society for Technical Education. 4) American Society of Civil Engineers India Section (ASCE-IS) 5) Maharastra Chapter of American Concrete Institute (ACI) etc. He has authored more than 90, well cited, journal and conference papers in leading International journals such as ACI materials Journal, Journal of Materials in Civil Engineering, ASCE; Cement and Concrete research and The Indian Concrete Journals and others, besides authoring state of the art report, lecture notes and articles in workshop proceeding etc. He also had a visiting position in EPFL Switzerland and involved in collaborative research programs with German Universities and University of Dundee UK. He has handled more than 200 consultancy and research projects and is actively involved in various national committees. Dr. Bhattacharjee has supervised 125 M. Tech. and 11 Ph.D. thesis till date and currently guiding 7 M.Tech. thesis work besides 10 ongoing Ph.D. research projects. He is also member of the editorial board of Magazine of Concrete Research and International Journal of 3Rs.