Indian Roads Congress
Special Publication 60

AN APPROACH DOCUMENT FOR ASSESSMENT OF REMAINING LIFE OF CONCRETE BRIDGES

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AN APPROACH DOCUMENT FOR ASSESSMENT OF REMAINING LIFE OF CONCRETE BRIDGES

BACKGROUND

The Bridge Maintenance and Rehabilitation Committee (B-9) in its meeting held on 4.7.97, constituted a Sub-group consisting of Dr. M.G. Tamhankar (Convenor), S/Shri S.S. Chakraborty, Ajit Singh, M.V.B. Rao, A.K. Harit and Mahesh Tandon as members for preparing draft report on Assessment of Remaining Life of Concrete Bridges. The draft document prepared by its Convenor, Dr. M.G. Tamhankar, was discussed by the Sub-group and in the B-9 Committee in its number of meetings. The final draft was approved by the B-9 Committee in its meeting held at Mumbai on 20.12.99. The personnel of B-9 Committee which approved this document is given below:

A.G. Borkar	..	Convenor
CE(R&B), NH, Hyderabad (D. Sree Rama Murthy)	..	Co-Convenor
D.K. Kanhere	..	Member-Secretary

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C.V. Kand	Gurdip Singh
P.Y. Manjure	Dr. M.G. Tamhankar
N.V. Merani	Mahesh Tandon

The Bridges Specifications and Standards Committee during its meeting held on 14th July, 2001 approved the document for printing as a Special Publication of the IRC subject to certain modifications in light of the comments made by the members. The draft was approved by the Executive Committee in its meeting held on the 16th December, 2001 and later by the Council of the IRC in its 164th meeting held at Kochi (Cochin) on the 8th January, 2002.

1. INTRODUCTION

With ever increasing stock of distressed bridges and dwindling resource position for maintaining the same, the need of assessment of remaining useful life of bridges in Bridge Management Systems cannot be over emphasised. It has been recognised that despite its crucial role the life assessment has eluded explicit modelling and numerical evaluation of its 'absolute' value due enormous complexities in the degradation mechanism, and lack of data-base on material degradation and bridge performance. The problem has got further compounded due to uncertainties associated with material properties, construction details, exposure conditions, deterioration rates and the quality of maintenance.

Therefore, to predict as to how long the given bridge will last is perhaps beyond the present level of development of the subject. At best one can try to estimate whether the given bridge can reach a specific age, with the help of probability distribution function of service life. This is also possible if the acceptable degree of failure probability can be pre-determined.

It has been emphasised[24] that our design philosophy comprises a number of essential and interacting elements, such as:

- a behavioural model,
- criteria defining satisfactory performance,
- loads under which these criteria should be satisfied,
- relevant characteristic material properties, which should reliably be achievable in the construction process,
- factors or margins to take account of vagaries and variability in the system.

But the present day predictive models, however, consider the first three of the above elements reasonably accurately. The relationship between materials forming the structure and the overall response of the structure is often so complicated that this indeed poses a question as to whether the service life prediction can ever be generalised.

However, given its crucial role in distressed or rehabilitated bridges, it becomes imperative to make a beginning in this direction. It is necessary to create awareness to keep track of performance against the assumed expectations in general and to look into specific issues, such as:

- existing load carrying capacity,
- cause of distress, if already noticed,
- degree of aggressiveness of the environment,
- risk of future damage,
- rate of deterioration, and
- the effect of repairs conducted from time to time on the rates of deterioration.

There is also an urgent need to document data, wherever available, on degradation causing factors, deterioration processes, damage modes, and deterioration rates. With these objectives in the background, this document aims at presenting the existing status of the subject by synthesising the available knowledge in the fields of material degradation and life assessment to initiate further steps and make a beginning towards introducing the concept of life prediction.

Given the developmental stage of this subject, the document is prepared as an approach document only, to enable engineers to appreciate the need of the subject, to understand present status of the knowledge and its limitations, and to introduce road map for future course of actions. Obviously the subject has not attained the level where it can be enforced as a part of normal design procedure.

1.1. **Durability Concept**

Durability is the property expressing the ability of the structure/component/product to maintain the required performance level over at least a specified time under the influence of the degradation factors. The minimum acceptable values for performance (or maximum acceptable values for degradation) are called durability limit states. The limit-state is a performance requirement critical to the service life, which can be set with regard to either the ultimate limit or the serviceability limit. A new feature here is the incorporation of time into the design problems. It, thereby, allows the possibility of treating degradation of materials as an essential part of structural calculations. Traditionally, the durability design is based on implicit rules for materials, material compositions, working conditions, structural dimensions, etc. (e.g., minimum concrete cover, maximum water/cement ratio, minimum cement content, crack width limitations, cement type, coatings on concrete, categorisation of exposure conditions).

1.2. Service Life

The concept of service life can be approached from at least three different aspects: (i) technical, (ii) functional and (iii) economic. The service life of a bridge span, component or its constituting materials is defined as the period of time after installation during which its essential properties meet or exceed minimum acceptable values, when routinely maintained. It is the period during which no excessive expenditure is required in operation, maintenance or repair. Each component has an expected service life. The exact definition of service life is obscured by the maintenance routines performed during the service life. Maintenance can influence the length of service life and hence, the definition of service life should include "when routinely maintained". Some components, such as, bearings, expansion joints, wearing coat, require periodic maintenance or replacement, whereas, the main structural components may also require periodic inspection and preventive maintenance but are expected to perform their expected functions during their service life. Required service life imposed by general rules, the client or the owner of the structure is called the target service life. Fig. 1 shows a generalised pattern of deterioration and maintenance cycles in the life of a bridge. Considering the corrosion-induced deterioration, life of the structure can be split into following four periods, Fig. 2.

- T_1 - time when the external agent (e.g., CO_2, chlorides) becomes active
- T_2 - time when the effect of CO_2/chlorides reaches steel zone (initiation time)
- T_3 - time when first visible sign of damage due to corrosion is noticed (e.g., crack, staining)
- T_4 - time when damage reaches serious level affecting structural safety (e.g., wider cracks, spalling), viz., end of the operational period
- T_5 - time when structure reaches failure stage.

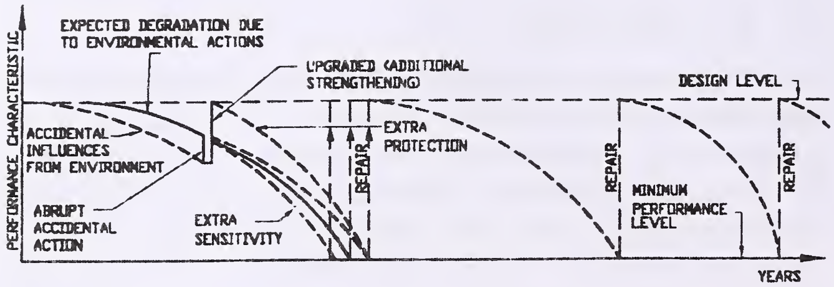

Fig. 1. Deterioration and maintenance life cycle[1,3]

According to the Danish Road Directorate[2], T_4, the operational period should be a minimum 100 years and T_3, a minimum of 50 years.

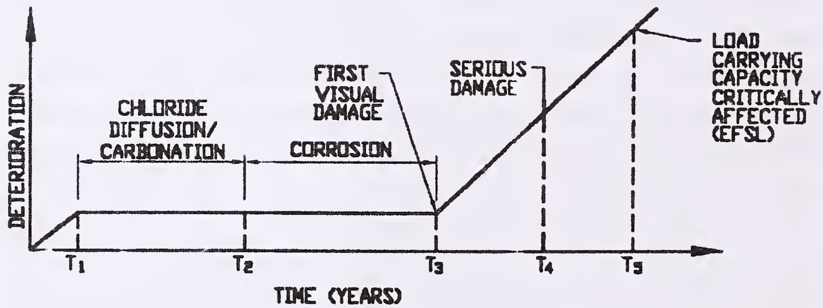

Fig. 2. Degradation cycle due to corrosion[2]

1.3. Design Life

Distinction needs to be made between Service Life and Design Life. The design life is a period considered (possibly longer than the service life) that will give sufficiently high probability of the structure achieving the required service life. The design life would depend upon the importance of the structure. It would ensure required safety against falling below the target service life. The design life would influence the specifications and detailing of structural components.

Various service/design life periods are prescribed in the literature according to importance, e.g.,

- Studies for a major suspension bridge over the Straits of Messina have indicated that the conventional service life of such structures should be 200 years.
- In the U.K., according to BS:5400, it is assumed to be 120 years.
- For the construction of structures for Oresund Link between Denmark and Sweden[2], the owners have specified a service life of 100 years, out of which the first 50 years must be totally free of maintenance, while minor concrete repairs will be acceptable after the first 50 years.
- Different components can have different design lives. Figures can vary from bridge to bridge as well as with wide range of structural systems and material qualities of components. Typical figures for different components are indicated in the following:

Foundation	:	Same as design life of the bridge
Piers & abutments	:	Same as design life of the bridge with periodic minor concrete repairs (e.g., after every 20 years)
Bearings	:	Varying depending upon type and quality
Steel main members	:	Same as design life of the bridge with periodic minor painting (e.g., after every 6 years) and major painting (e.g. after every 12 years)
Concrete decking	:	Same as design life of the bridge
Wearing coat	:	Varying depending upon type (e.g., 15 years)
Expansion joints	:	Varying depending upon type and quality (e.g., 15 years)

Euro-code 1 (CEN-1994) presents the classification for the target service life as shown below[44]:

Class	Target Service Life (Years)	Example
1.	1-5	Temporary structures
2.	25	Replaceable structural parts, e.g., bearings
3.	50	Building structures and other common structures
4.	100	Monumental building structures, bridges and other civil engineering structures

In fact, it is not feasible to design a bridge for a very specific life period. The available experience of traditional materials is often inappropriate and many modern high technology products are relatively untried. Hence, the anticipated durability can only be an estimate, its prediction being subject to many variables. In general, the designer has in mind a useful life for the bridge alongwith reasonable level of maintenance from time to time.

2. DEGRADATION CAUSING FACTORS, DETERIORATION PROCESSES AND DAMAGE MODES

The process in which the resistance (and thereby performance) decreases with time is called degradation process. Materials and components have finite service lives since they gradually undergo chemical, physical or mechanical changes resulting in degradation and reduction in their ability to perform as required. Degradation processes are numerous. The type and the rates of deterioration processes determine the resistance and rigidity of the materials, sections and the elements constituting the bridge structure.

2.1. Degradation Causing Factors

Commonly recognised degradation causing factors can broadly be categorised into the following:

(i) **Water**

- Liquid (e.g., rain water/stagnating/trapped/splashing zone, etc.)
- Solid (e.g., snow, freezing of water)
- Vapour (e.g., air humidity, moisture in pores)

(ii) **Aggressive air constituents**

- Carbon dioxide (leading to carbonation)
- Chlorides (promoting corrosion)
- Sulphates (leading to expansive reaction with cement)
- Acids (dissolving cement)
- Alkalies (leading to expansive reaction with aggregates)
- Ozone
- Ammonia (leading to disintegration of concrete)
- High voltage transmission lines (stray currents)

(iii) **Biological agents**

- Micro-organism
- Fungi

(iv) **Temperature**

- Absolute value
- Gradient
- Cycle
- Solar radiation

(v) **Foundation condition**

- Scour around piers and foundations
- Sub-soil strata leading to settlement, tilt, etc.

(vi) **Time dependent material characteristics**

- Creep in concrete
- Relaxation in steel

(vii) **Traffic**

- Axle loads (leading to abrasion, impact or exceeding design load limits)
- Frequency (leading to fatigue)

(viii) **Other non-continuous actions**

- Earthquake
- Cyclone
- Collision with vehicle, barge, etc.
- Vandalism

Climatic influences on bridge components can be further sub-divided into macro, meso and micro climates, corresponding to country, site and specific element of the structure respectively. Depending upon the nature and duration of the degradation causing factors and the type of the bridge, it will lead to partial or complete loss of serviceability or of its mere aesthetic appearance.

2.2. Degradation Processes/Mechanisms

Bridge component fails, if the strength of that component is no longer sufficient to resist the actual load effects. The resistance of the component is affected due to various degradation processes to which the material of the bridge component is exposed. The sequence of chemical or physical changes that lead to detrimental changes in one or more properties of structural materials or components when exposed to one or a combination of degradation factors, is called Degradation mechanism. The degradation processes follow different patterns as listed below:

(i) Degradation processes progressing linearly with time, e.g., corrosion process over years, wear and tear of the deck.
(ii) Degradation processes slowing down with time, e.g., carbonation in concrete.
(iii) Exponentially accelerating degradation process, e.g., fatigue.
(iv) Degradation process non-continuous in time, e.g., collision, earthquake, etc.

(v) Two-stage mechanism of degradation process, e.g., degradation of protection layer first as in the case of cover on reinforcement, or as in the case of coating on a steel member.

Fig. 3 depicts the above patterns of degradation. Some of the commonly observed deterioration processes are the following:

2.2.1. **Fatigue :** It is defined as the tendency of the material to break under repeated cyclic stresses considerably below the ultimate tensile stress. Cyclic loading induces or propagates the pre-existing cracks in the bridge. Movement of frequently occurring heavily loaded commercial vehicles can cause fatigue damage in road bridges. The fatigue action comprises three phases: (a) crack nucleation in which invisible changes occur, (b) crack propagation in which micro-cracks grow to visible macroscopic dimensions, and (c) instant failure.

2.2.2. **Corrosion of steel :** Bridges in a coastal belt have shown premature distress due to corrosion of steel arising from salinity in the air. Corrosion of steel in concrete girder can be faster than that in the steel in the deck slab due to crowding of reinforcement in girders especially at the lap joints with accompanying honey-combing.

Corrosion is the mechanism, which is in most cases an important parameter to influence the load-carrying capacity and the service life of the bridge. It can lead to: (a) loss in steel integrity, (b) possible loss in mechanical properties of steel, strength and ductility, (c) spitting and spalling of the concrete cover and possible loss of effective concrete cross-section, (d) loss of bond between steel and concrete in case of cracks running parallel to the steel, and (e) loss of prestress in the case of corrosion of pre-stressing steel. Pre-stressing steel apart from uniform corrosion may also suffer from pitting corrosion, crevice corrosion, stress corrosion and hydrogen embrittlement.

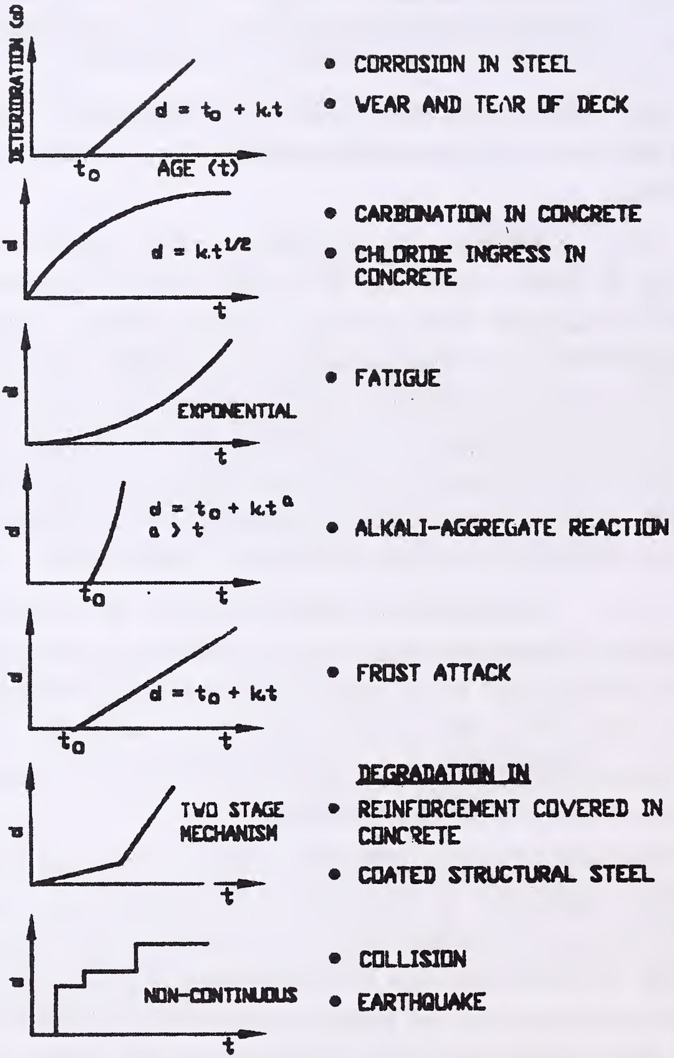

Fig. 3. Simplified deterioration models[4, 8]

2.2.3. **Disintegration, cracking, delamination of concrete** : These are the commonly observed distresses in concrete leading to loss of stiffness, composite section, etc.

2.2.4. **Combination effect (Synergy) :** In actual service, degradation factors may interact to increase the rate of degradation. Synergistic actions are in fact difficult to simulate or to account for, e.g., the effect of corrosion induced defects on the fatigue strength of steel bridges, or influence of temperature on creep in concrete bridges.

In the presence of aggressive environment causing corrosion of embedded steel as well as deterioration of concrete, the fatigue life of the structural component can get reduced considerably. The phenomenon is termed as corrosion fatigue. It is reported[32] that fatigue lives of beams exposed to sea-water are fifty per cent less than those in air.

Initiation and later the development of corrosion can also get accelerated by alkali-aggregate reaction (AAR) and frost impact. Furthermore, AAR and frost taking place at the same time may accelerate each other[39].

2.3. Damage Modes

The degradation causing factors have different effects on the structure, leading to different damage modes. A distinction has to be made between the disintegration at the micro level in the material and the collapse mode of the bridge from the structural consideration. Degradation mechanism can be discovered at an early stage in some situations by measuring - potential differences, settlement, sagging and strains, density and thickness of concrete cover, depth of carbonation and chloride penetration, etc. But the severity of the damages needs to be examined from their range of influence, e.g., (i) loss of section, (ii) limited to failure of structural element, (iii) failure of the entire structure. In this exercise, weightage needs to be given to the degree of risk involved. Risk is defined as the product of failure consequence and failure probability. A minor fault in a critical part of the bridge may signify a greater risk than a more extended fault in a less critical area.

3. DETERIORATION RATES

The correlation between the attacking agent and the quantum of deterioration with time is the first requirement in prediction of the remaining life. However, there are considerable difficulties in modelling deterioration or in collection of past data on performance leading to extrapolation of available stray information. Despite the on going research world wide, exhaustive analytical models to predict rates of deterioration in different components of the bridge are not yet available for their application in practical cases. Models are also not available to consider interaction of aggressive actions that can act simultaneously. Deterioration rates suggested by various researchers are discussed in the following paras.

3.1. Deterioration Rate Due to Carbonation

Carbonation is the reaction of CO_2 in the air with hydrated minerals in concrete, leading to lowering of the pH value in the carbonated zone. The protective film on the surface of the steel lying in the carbonated zone is destroyed and the propagation of corrosion starts. Time for carbonation front to reach reinforcement leading to initiation of corrosion is called initiation time. The rate depends upon cover thickness, cement, curing, density, water cement ratio, etc. Carbonation penetration below the concrete surface progresses as the square-root of time. Thus, if concrete cover is doubled, corrosion of reinforcement is delayed by four times. By the same token errors in fixing reinforcement which may reduce cover can affect the steel earlier than expected. Expressed mathematically[44],

$$\mu\,(d) = K_c t^{1/2}$$

Where,

$\mu\,(d)$	is the mean of the depth of carbonation at time t (mm)
K_C	is the carbonation rate factor (mm/year$^{1/2}$) and
t	is the time (or age in years)

The initiation time of corrosion t_0 can hence be expressed as

$$t_0 = [d/K_C]^2$$

Where,

d is the concrete cover

The carbonation rate factor K_C depends upon the strength and composition of the concrete, and on environmental factors, like, humidity, temperature, etc. The depth of carbonation is assumed normally distributed and the coefficient of variation (ratio of standard deviation to mean) is assumed to be constant. Fig. 4 shows the process of carbonation in the concrete cover.

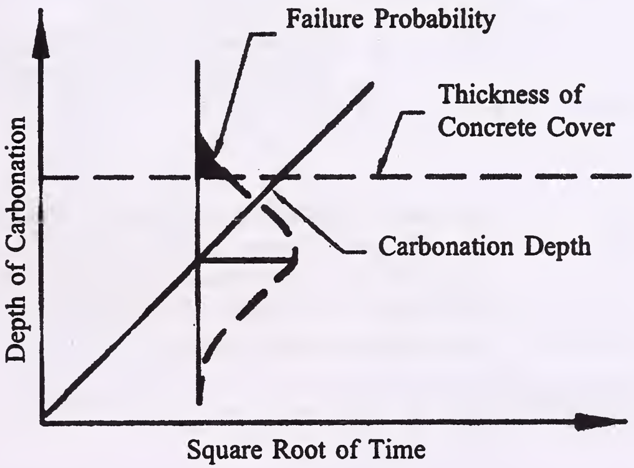

Fig. 4. Schematic representation of the process of carbonation within a concrete cover[44]

The part of distribution of carbonation depth that exceeds the thickness of the concrete cover shows the failure probability. There are several formulae for modelling of carbonation rate

K_C as follows[44] :

(i) $K_c = \{C_{env}\ C_{air}\ a\ (f_{ck} + 8)^b\}$

Where,

C_{env} is the environmental coefficient
C_{air} is the coefficient of air content
f_{ck} is the characteristic strength of concrete (MPa)
a, b, constants (depending upon the binding agent)

Suggested values for C_{env}, C_{air}, a and b are

C_{env} = 1 (structures sheltered from rain)
= 0.5 (structures exposed to rain)
C_{air} = 1 (not air entrained)
= 0.7 (air entrained)
a = 1800, b = - 1.7 (for Portland Cement)
a = 360, b = - 1.2 (for Portland Cement with 28 per cent fly ash or 70 per cent blast furnace slag)

(ii) $K_c = \{\ [\ 2\ D_c\ (\ C_1 - C_2)\ ]\ /\ a\ \}^{1/2}$

Where,

a = the amount of alkaline substance in concrete
D_c = the effective diffusion coefficient for CO_2 at a given moisture distribution in the pores (m^3/s)
$C_1 - C_2$ = concentration difference of CO_2 between air and the carbonation front (Kg/m^3)

(iii) $K_c = [64\ k^{0.4}]\ /\ C^{0.5}$

Where,

K = oxygen permeability of concrete at 60 per cent RH
c = alkaline content in the cement

As a typical example for the following data, the distribution function of probability of failure due to carbonation, and probability density function are presented in Figs. 5 and 6 (refer para 4.4.2).

Data:

- Structure sheltered from rain (C_{env} = 1.0)

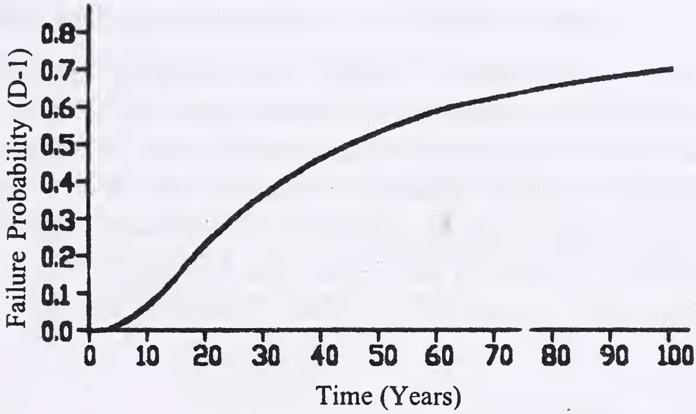

Fig. 5. Probability distribution function of service due to caribonation[44]

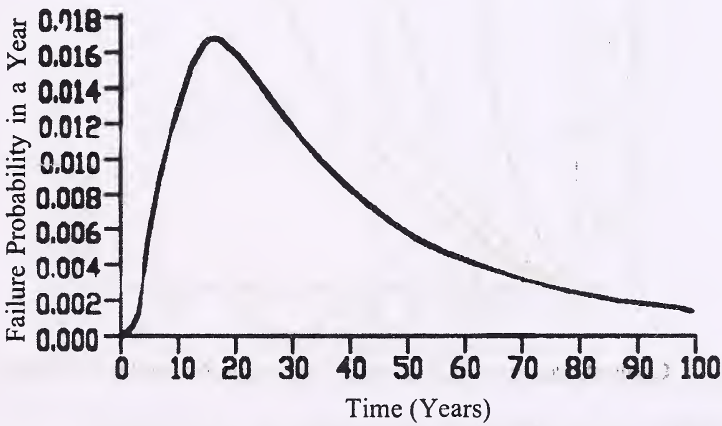

Fig. 6. Probability density function of service life due to carbonation[44]

- Concrete made with portland cement with no air entrainment (C_{air} = 1)
- Characteristic compressive strength of concrete 30 MPa (f_{ck} = 30).
- Thickness of cover 25 mm (D = 25).
- Coefficient for variance for carbonation depth (v = 0.6)

- Coefficient for variance for concrete cover (v = 0.2)
- Constants a=1800, b= -1.7 (applicable for portland cement)

Fig. 7 is the result of studies[3] indicating the age to start corrosion due to carbonation in various grades of concrete. Yet another study[7] observes that carbonation and chlorides can penetrate to the interior of concrete at a lower rate than would be given by a square-root of time function. This means that if the concrete cover is halved, the critical state for incipient danger of corrosion will be reached in less than a quarter of the time.

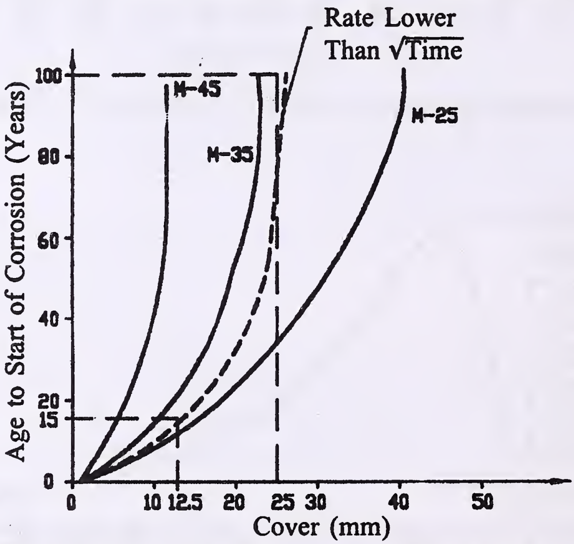

Fig. 7. Concrete cover *vs.* time to start carbonation induced corrosion[3,7]

3.2. Deterioration Rate Due to Corrosion

Corrosion of reinforcement in reinforced concrete members can lead to rust strains on the concrete surface, cracks in the cover, spalling of concrete and the loss of bond between steel and concrete. The action is complex since apart from general uniform corrosion, there can be localised corrosion with pit depths as much as five to ten times deeper than the average corrosion penetration.

Concrete protects steel physically and chemically. It provides physical barrier against agents that promote corrosion, e.g., water, oxygen, chlorides. The chemical effect of concrete is attributed to its alkalinity, causing oxide layer to form on the steel surface. The deterioration due t_o corrosion consists of two time segments: (i) Initiation Time to i.e., the time required for external agents to de-passivate the protective oxide film on steel bars, and (ii) Propagation time t_i i.e., corrosion action leading to rust formation, reduction in bar diameter, cracking of concrete, loss of bond, etc. (Fig. 8). As a conservative approach it is advisable to limit the service life of the member corresponding to initiation time only. Governing deterioration rates in such cases would be those due to actions of causing de-passivation of steel.

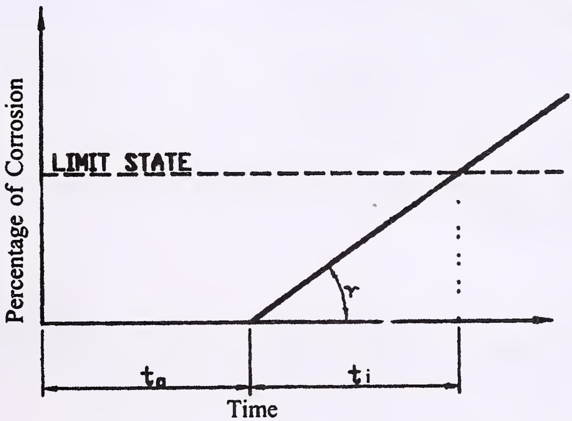

Fig. 8. Determination of service life with respect to corrosion of reinforcement

Propagation time begins when passive film is destroyed as a result of falling pH due to carbonation, or as a result of chloride content rising above the threshold close to the reinforcement. Some guiding expressions are available in the literature to determine the propagation time. However, it should be noted that the rate of corrosion of steel in cracks or in the presence of chlorides is not yet fully understood.

Several techniques are being explored to detect presence of corrosion activity. Techniques involving visual inspection, detection of laminations, measurement of corrosion potentials or the extraction of physical samples for laboratory analysis is generally both time-consuming and expensive. Rapid and non-contact type techniques, such as, impulse radar and infrared thermography may prove useful. But although these techniques would detect delamination that may be due to corrosion of reinforcement, they do not detect corrosion directly. Quantitative assessment of corrosion activity in concrete is still in the research stage. Following are the formulae for estimating propagation of corrosion[44]:

(i) $$t_l = \frac{\Delta R_{max}}{r}$$

(ΔR_{max} based on critical threshold value of the structural capacity of the member)

Where,

t_l = the propagation time of corrosion (years)
ΔR_{max} = the maximum loss of radius of the steel bar
r = the rate of corrosion (μm/year)

(ii) $$t_l = \frac{80C}{D.r}$$

(based on cracking of the concrete cover)

Where,

C = the thickness of the concrete cover (mm)
D = the diameter of the reinforcement bar (mm)

Rates of Corrosion

The rate of corrosion in concrete required in the above expressions depends strongly on the environmental factors, such as, temperature and humidity, besides chloride contents. Influence of temperature is considered by the following expressions:

$$r = C_T r_o$$

Where,

C_T = the temperature coefficient and,
r_o = the rate of corrosion at + 20°C
(values of CT recommended for European Countries vary between 0.21 and 0.73)

Influence of relative humidity in carbonated and chloride-contaminated concrete is illustrated in the following[44]:

Relative Humidity (%)	Rate of corrosion in	
	Carbonated concrete (μm/year)	Chloride contaminated concrete (μm/year)
99	2	34
95 (exposed to rain situation)	50	122
90 (sheltered from rain situation)	12	98
85	3	78
80	1	61
75	0.1	47
70	0	36
60	0	19
50	0	9

The above values are approximate averages based on the experimental data reported by Tuutti.

Yet another study recommends the following values for the mean corrosion rates in the calculations:[44]

Action	RH (%)	Corrosion rate (μm/year)
Carbonation only	90 - 98 <85	5 - 10 ≤ 2
Chloride contamination	100	≤ 10
	80-95 <70	50-100 ≤ 2

iii) Linear Polarisation Resistance (LPR) technique[46]

This method developed for on-site study of corrosion rates of steel in concrete, is based on the experimentally observed assumption that the polarisation curve for a few mV around the corrosion potential obeys quasi-linear relationship. The slope of this curve is the polarisation resistance R_p where

$$R_p = \frac{\Delta V}{\Delta I} \text{ as } \Delta V \rightarrow 0$$

From this slope the corrosion rate is determined using relationship

$$i_{corr} = B/R_p$$

where i_{corr} is the corrosion current or corrosion rate (μA/cm²) and B is a constant the value of which lies between 13 and 52 mV. B equal to 40 is considered to be adequate in many cases. A typical plot of linear polarisation resistance curve is shown in Fig. 9.

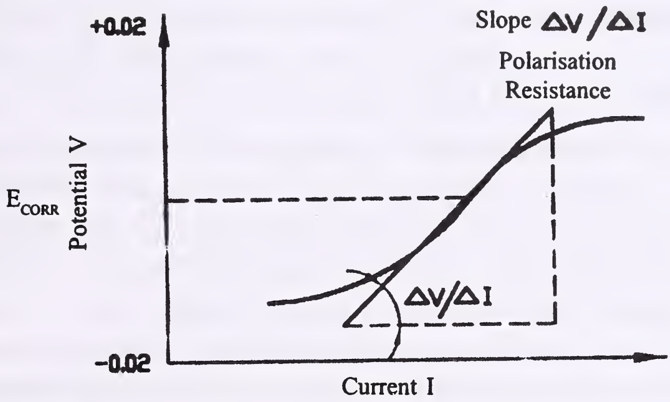

Fig. 9. Typical plot of polarisation resistance[46]

Having obtained $i_{corr,}$ the rate of degradation in reinforcement bar due to corrosion can be expressed as,

$$\phi_t = \phi_i - 0.023\ i_{corr}\ t$$

Where, ϕ_t is the reinforcement bar diameter at time t (mm), ϕ_i is the initial diameter of reinforcement bar (mm), i_{corr} t is the corrosion current or corrosion rate (µA/cm²), t is the time after beginning of the propagation period (years), 0.023 is the conversion factor of µA/cm into mm/year. From above expression, it can be deduced that the corrosion current of 1 µA/cm², measured by resistance polarisation can be expected to lead to corrosion penetration into iron of 11.6 µm/year[10].

The above formula for degradation in reinforcement bar has been further modified[35] by multiplying i_{corr} t by two more parameters 'W_t' and 'a' to include localised corrosion effect, since pitting corrosion can affect smaller diameter bars significantly. Here 'W_t' is the wetness period or effective time of corrosion and 'a' is the concentration factor due to localised corrosion (varying from 2 to 10).

In order to determine the impact of this loss of steel section on the structure, it is necessary to distinguish between the serviceability and the ultimate limit states. While the loss of diameter of steel section would affect the ultimate flexural strength, much before that cracking of concrete cover due to corrosion would determine the permissible threshold value for corrosion penetration from serviceability and durability considerations. A value of about 150 μm is generally considered the threshold value for corrosion penetration to cause cracking of concrete. Based on this value and the deduction arrived at from the above corrosion rate (i.e., i_{corr} of 1 μA/cm² leading to 11.6 μm corrosion penetration per year), following guidelines are available[8]:

$i_{corr} < 0.22$ μA/cm²	no corrosion damage exected.
$0.22 < i_{corr} < 1.08$ μA/cm²	corrosion damage possible in the range of 10-15 years
$1.08 < i_{corr} < 10.8$ μA/cm²	corrosion damage expected in 2-10 years
$i_{corr} > 10.8$ μA/cm²	corrosion damage expected in less than 2 years.

Above guidelines need to be revised with more experience and as the data becomes available. For example, with respect to cracking of the cover which results from the expansive character of the iron oxides, it has been reported[36] that around 10-50 microns of corrosion penetration is enough to produce visible (0.05 mm wide) cracks. However, accuracy of prediction would depend upon the estimation of the value i_{corr}. Further, the technique may be suitable in RC components but may not help in detecting damage to prestressed steel. For prestressing steel in view of their smaller diameters, initiation time alone should be considered to determine the onset of corrosion.

3.3. Deterioration Rate Due to Ingress of Chloride

Because of exposure to the salt water in the coastal regions, concrete bridges are contaminated with chlorides. The presence of chlorides (resulting in the loss of alkaline environment), oxygen and water lead to corrosion of embedded steel. Chloride ions diffuse through the porous concrete and reach the corrosion threshold value. The resultant cracking of concrete cover allows intrusion of chlorides and oxygen at a much faster rate, thus accelerating the corrosion process. Existing chloride in concrete is not considered in the following expression. As a result of chloride penetration a gradient is set up near the concrete surface. The time at which the critical chloride content reaches the steel surface and de-passivates it, is regarded as the initiation time of corrosion.

Estimation of service life in this case is based on FICK's second law on diffusion of chloride ions through porous materials such as, concrete[18]. To model the chloride transport process in a porous material, it is assumed that a saturated condition exists and that, FICK's law applies, even though corrosion does not occur when the material is continuously saturated because of lack of oxygen for cathodic reactions. In reality, there is a combination of exposure conditions. Further, one of the assumptions in the derivation of FICK's 2nd law is that the porous medium is homogeneous, which is not the case for concrete. It is also assumed that the medium is non-reactive and non-absorptive, and that does not hold for concrete either. Chloride ions can be physically absorbed on to the surface of the pores and chemically combined to the aluminates. Furthermore, experimental testing in concrete specimens has shown that the diffusion coefficient varies with time, solution type and concentration. The time dependence is, in part, a direct consequence of the continuing cement hydration reactions and pore blocking. Despite the differences between the assumptions, FICK's law still provides the only way available to model chloride diffusion into concrete[47]. It is expressed as

$$C(X, t) = C_0 \{1 - \mathrm{erf}\,[(X/2)/(\sqrt{D_c} \cdot t)]\}$$

Where,

$C(X, t)$	=	chloride concentration (kg/m^3) at depth X at time t
C_o	=	equilibrium chloride concentration (kg/m^3) (assumed 1.3 cm below the surface)
erf	=	error function
D_c	=	chloride diffusion constant (property of concrete cm^2/year)
t	=	time in years (for initiation time, X = cover of concrete)
X	=	depth at which chloride concentration is required (i.e., depth where steel is located)

Thus, the initiation time is calculated using above expression on the basis of chloride measurements carried out on the bridge, and/or from measurements of carbonation. Instead of modelling chloride ingress gradient by error function, the formula is simplified by using a parabolic function as follows[44]:

$$C(X,t) = C_0\left(1 - \frac{X}{2(3D_c t)^{\frac{1}{2}}}\right)^2$$

As an illustration, Fig. 10 gives the theoretical service life as a function of D_c (Diffusion coefficient) and cover for C_s (chloride content as per cent by weight of dry concrete)= 0.2 per cent and 0.1 per cent[2]. Diffusion coefficient D_c is calculated as equal to 5000 * $(W/C)^5$ mm/year. Limited data is available since it is either based on laboratory experiments or from practical data collected from existing structures of approximately same age group (20-30 years old) only. General validity of the model for use in long term predictions has, therefore, not been confirmed in practice. Sensitivity of parameters C_s and D_c can be seen from Fig.10[2]. If C_s is not 0.2 per cent as estimated but 0.4 per cent, the service life is theoretically reduced from 40 to 30 years. If D_c is not 40 mm/year but say 80, service life is theoretically reduced from 40 to 28 years[2].

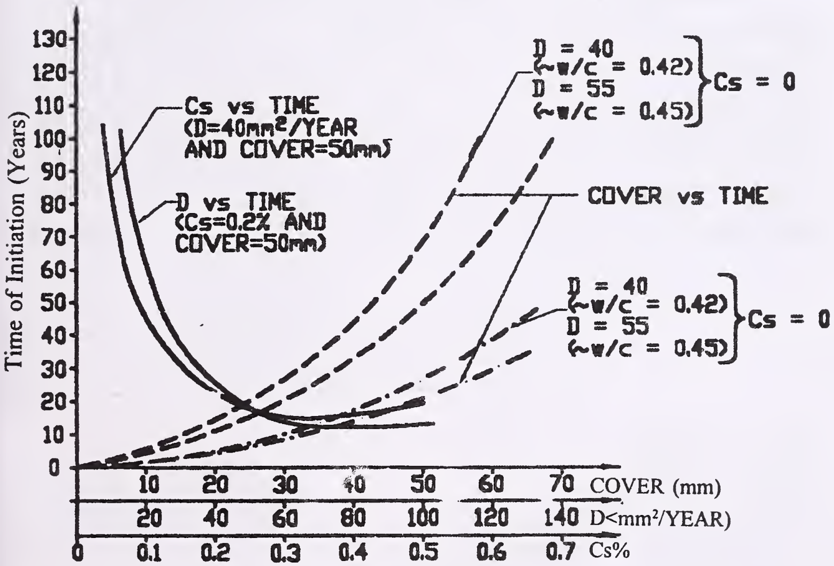

Fig. 10. Relationship between cover, diffusion coefficient (D), chloride content (C_s) and time of initiation[2]

Estimation of service life in the case of chloride attack on piers of five different bridges is illustrated[39], ***(Appendix).***

3.4. Scale for Deterioration Rate

Deterioration rates discussed earlier relate to micro-level degradation inside the body of the structure. Different ratings for damages or for performance of the bridge component have been proposed in the literature.

RILEM draft recommendations for damage classification of concrete structures[41] indicate ratings for different visible damages. ECB bulletin[10] classifies different levels of deterioration states arising from damages due to corrosion based on external signs, such as, rust spots, cracks, cover spalling and reduction in the reinforcement cross-section.

Under the FHWA inspection procedure[9] condition of bridge elements, such as, deck, superstructure or substructure is evaluated on a scale of 9 to 0 (9 being the perfect condition and 0 the worst). Thus, when the bridge element rating changes from one condition rating to another, it can only change in integer values, such as, 1, 2, etc. The data for condition rating versus time, therefore, do not yield a smooth curve when plotted (Fig. 11). Unless the

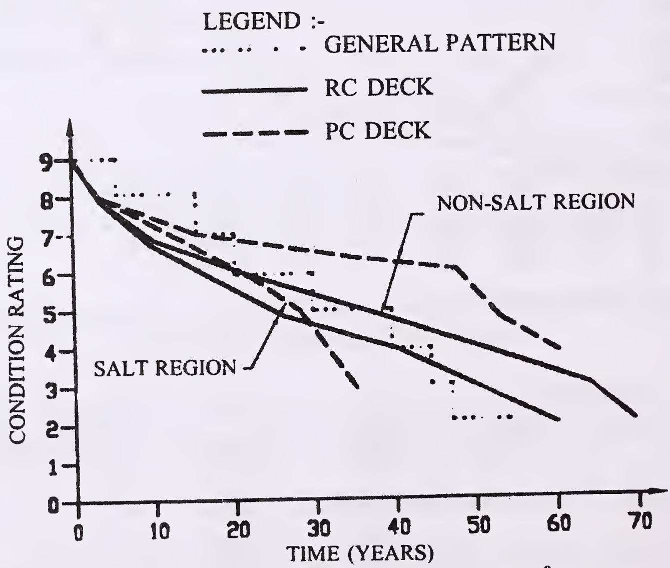

Fig. 11. Deck condition rating *vs* time[9]

maintenance or rehabilitation is performed the element condition rating is expected to remain unchanged or to drop downward on any subsequent inspection, which is normally conducted once in every two years by trained technicians. Several studies incorporating the above condition rating scale have been reported from the U.S. Although the observations from these studies cannot be directly extended in a generalised manner, a few of the observations extracted below can be of interest.

A study[9] has indicated that the structural condition of the deck would deteriorate at the rate of 0.094 per year during the first ten years and thereafter at 0.025 per year. This implies that the average condition would never fall below a condition rating of 6 until after 60 years.

Another study[9] indicated that the deck would deteriorate slightly faster with age than the superstructure or substructure. The study estimated the average deterioration of decks to be about 1 point in 8 years and that of both superstructure and substructure to be about 1 point in 10 years.

In yet another study[22], bridge superstructure deterioration was found to be a convex function, with superstructure condition ratings deteriorating more slowly as the bridge ages. Fig. 11 indicates the typical deterioration curves for bridge decks of RCC and PSC[9].

4. METHODOLOGIES FOR LIFE PREDICTIONS

In the case of a bridge under investigation, a clear understanding of the agencies which can cause decay and the manner and extent to which they will have an effect, singly or in combination becomes a pre-requisite. Obviously, fewer the 'ageing causes' (e.g., only carbon dioxide or fatigue), easier becomes the handling of life estimation. Synergistic effects are complicated and unpredictable. The situation becomes worse if accidental actions are also to be considered along with environmental aggressivity. Further for prediction of remaining life, in the absence of data on construction defects, it may become necessary to assume that the bridge has been designed and constructed as per the known provisions in codes and specifications.

To determine service life, it is necessary to model bridge deterioration process. A number of methods have been proposed, although most of them are still in the developmental stage. They are summarised in the following:

4.1. Estimates Based on Experience

This method relies primarily on engineering judgement and past experience of the investigator.

4.2. A Comparative Approach

For estimating life of specific material, deductions are made from the performance of similar quality materials in similar exposure conditions. One approach is, therefore, to record condition states of a particular bridge element from similar bridges of different age groups, so that a graph of degradation pattern against time can be plotted. This would help in estimating rate of degradation and thereby to predict the remaining life of the element in question.

The following are some of the potential determinants of bridge superstructure deterioration[22].

- AGE
- Average daily total volume of traffic (ADT)
- Average daily total volume of truck traffic (ADTT)
- Bridge structural material
- Structural type
- Span length

- Maintenance level
- Environment

This approach, therefore, assumes that above data in addition to the periodic condition rating assigned by the evaluator to the bridge components, such as, deck, superstructure, sub-structure, etc. is available. Regression analysis is used in such cases to estimate parameters, which describe a functional relationship between empirically measured sets of dependent and independent variables. Here, condition rating becomes the dependent variable and the factors leading to degradation, such as, age of concrete, live load intensity, etc., are the independent variables.

For example, one such regression based equation obtained in one study[12] where AGE and ADT were considered the likely primary determinants of superstructure deterioration, is:

$$\text{Superstructure condition rating} = 9.0 - 0.674\,(\log \text{AGE}) - 0.005\,(\log \text{ADT}) \quad \text{(for steel)}$$

$$= 9.0 - 0.444\,(\log \text{AGE}) - 0.024\,(\log \text{ADT}) \quad \text{(for prestressed concrete)}$$

Where, AGE is the age of the bridge in years, and ADT is the average daily traffic. (The ADT is calculated as per item 29 of the National Bridge Inventory of the Federal Highway Administration[22]). With the help of such expressions, the age corresponding to the lower end of the rating scale would indicate the threshold of the service life.

Problems in comparative approach

(i) One has to be cautious in utilising data from different bridges. Sometimes the failures/degradations are caused due to incorrect design, poor construction work or faulty maintenance. The field data from such sites may generate important information on the quality of design, construction and maintenance but does not necessarily give the required feedback of information on performance of materials.

(ii) In regression analysis finer the homogeneity of the bridge group, better could be the accuracy of the regression equation.

(iii) Accuracy of the regression equations hinges upon accuracy of the initial data, in particular, the rating assigned by the investigator after the inspection. This in turn would depend upon the NDT, instrumentation and judgement of the investigator.

(iv) The problem arises with many of the materials used today, such as, admixtures, protective coatings which have a relatively short performance history.

Reliable data can, therefore, be generated from field performance only if the data stems from well planned systematic inspections of the state of thoroughly characterised existing bridges in thoroughly characterised environments.

4.3. Accelerated Testing

Simulated testing, properly correlated with service conditions can enable life predictions with reasonable confidence. Accuracy in prediction would logically depend on how accurately the environmental agencies that produce changes in the materials are defined and simulated in the tests. It is mainly because of lack of knowledge of the quantitative levels of the agencies likely to cause deterioration that researchers are tempted to test samples exposed in the field. This, however, being an extremely slow process, one resorts to an accelerated version in the laboratory. Here an assumption is made that the number of cycles in the accelerated ageing test have some kind of relationship to the life time of that material in actual condition. Before designing the accelerated laboratory test, it is, therefore, necessary to understand degradation mechanism so that in the test only those factors of degradation are incorporated which are most influential.

Fatigue testing of structural steel of railway bridges in the laboratories is a typical example of accelerated testing[11]. The prediction of fatigue life in these bridges is based either on (a) cumulative damage assessment based on S-N curves or (b) cumulative crack increments based on Fracture Mechanics. Both approaches require assessment of cumulative fatigue damage because fatigue behaviour under constant amplitude loading can be markedly different than that under variable amplitude loading.

The prediction of fatigue life through S-N curves comprises following activities:

- In-situ strain measurements at critical locations
- Theoretical corroborative analysis
- Live load surveys
- Dynamic signal analysis
- Preparation of stress histograms
- Fatigue tests in the laboratory
- Plotting of S-N curves
- Estimation of rate of fatigue damage
- Assessment of remaining life

4.3.1. **Stress-history :** This involves instrumentation of the bridge and measurement of strain-time history at critical sections, Fig. 12. Corroboration of stress intensities through analytical modelling is desirable. Traffic surveys and old records help to estimate the number of cycles of different stress ranges to which the bridge components are already subjected. Possibility of future increase in the traffic intensity also needs to be considered.

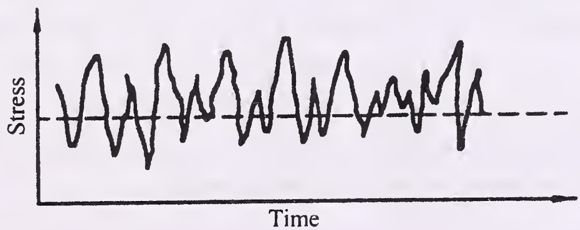

Fig. 12. Variable amplitude stress - history

4.3.2. **Stress-histograms :** Stress-history traces from the field are analysed to construct stress-histograms showing the number of cycles for different stress ranges by using one of the counting methods, such as, Rain-flow counting, or any other suitable method[11]. The choice depends upon the type of signal to be analysed. In these methods, time element is not taken into account. FFT analysers process a random dynamic signal on automatic basis. Similarly by making use of analogue to digital converters and dedicated software, dynamic signals are analysed to obtain stress ranges and their corresponding number of occurrences.

4.3.3. **Fatigue testing in laboratory :** Fatigue tests are conducted on representative samples (about 6 to 8) from the bridge for each stress range. Programmable loading facilitates better simulation of the actual load. Tests are continued till the failure of the specimen or upto two million cycles. Few specimens are also tested to assess the physical and chemical properties of the material, so that fatigue test data from similar materials can also be taken into consideration. If the number of specimens tested is small, S-N curve is plotted through test points taking geometric mean of all the test results for each stress-range and assuming that the curve follows a straight line between $N = 10^4$ and $N = 10^6$ cycles. If median value of test results for each stress-range is plotted against the number of cycles to failure, then this relationship would provide 50 per cent probability of fatigue failure for the member, Fig. 13. This means that at particular stress range value S1, 50 per cent of the samples will have failed before reaching the cycles N_i. For more exact determination of the P-S-N curves where P is the probability of the specimen's survival, a large number of specimens need to be tested.

4.3.4. **Estimation of damage accumulation and remaining life:** Although, most fatigue tests are conducted at

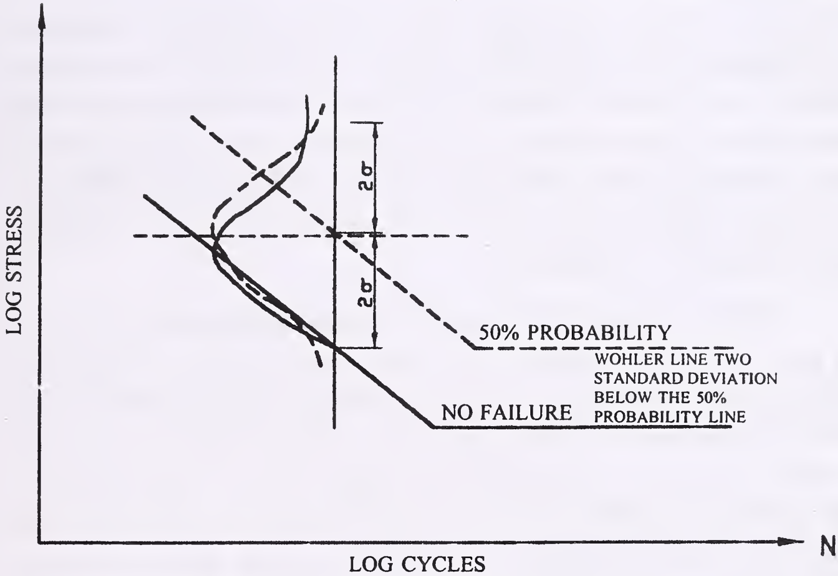

Fig. 13. 50 per cent probability S-N curve[11]

constant amplitude of cyclic stress, in reality the bridge components receive a load spectrum, i.e., the load and cyclic stress vary in some fashion under service conditions. To consider this feature, it is assumed in Miner's hypothesis that the damage by fatigue action accumulates before failure.

As per Palmgren-Miner's hypothesis, if N_i cycles of constant amplitude stress cause failure, then n_i cycles of the same stress range use up a fraction of (n_i / N_i) of the life. Failure would occur when the sum of used life fractions D_f reaches unity, i.e.,

$$D_f = \Sigma\ (n_i / N_i) = 1$$

Numerous variable amplitude loading tests have shown that sum of the average cycles at failure, called the damage sum at failure D_f, may deviate considerably from unity. The main reason for the deviation of the damage sum (D_f) from unity is that fatigue damage is assumed to be a linear function of $\Sigma\ n_i/N_i = 1$, irrespective of the loading sequence and stress level. But non-linear damage accumulation, effect of residual stresses and some other interaction effects can result in conservative and non-conservative deviations from Miner's fatigue criterion and in different values of the damage sum. Therefore, researchers recommend that 50 per cent probability of fatigue failure curves (S-N curves) and Palmgren-Miner's summation of 0.3 in place of 1.

Miner's damage accumulation index (D_i) caused per day or per vehicle is computed from the equation, $D_i = \Sigma\ (n_i / N_i)$, where n_i is the number of stress cycles actually applied and N_i are the number of stress cycles to cause failure. Here the number of cycles n_i for each stress range level is obtained from the stress-histograms of the component and the corresponding values of N_i are obtained from the S-N curve. The remaining fatigue life is computed by dividing-D (Miner's damage index) with fatigue damage caused per day (D_i).

D is taken equal to one at failure, if the S-N curve for 1 per cent probability of fatigue failure is used for predicting partial damage per day. Test results have, however, shown that the sum of cycle ratios (n/N) defers widely from the value of unity. In case S-N curves with 50 per cent probability of fatigue failure are used to compute damage per day. Damage Index D is taken as 0.3 at failure. Fatigue behaviour forms the distinct branch of research with a large specialised literature[11, 48-50].

Problems in accelerated testing

The difficulties experienced in utilising the accelerated tests are the following:

(i) Along with the accelerated test, it is necessary to check how well the results of this accelerated ageing test compare with those from an in-service exposure. But this comparison is seldom definitive, more often the correlation is only marginal.

(ii) Most of the test methods that have been developed for generating service life data focus upon climatic agents as the factors causing degradation. These climatic agents are difficult to quantify and to incorporate meaningfully into accelerated tests.

(iii) Short term accelerated ageing tests are usually designed to evaluate the effect of a small number of degradation factors. Although such test results may be useful for ranking of materials, they are only of limited value for predicting service life, unless the degradation factors studied are those that are responsible for all of the in-service degradation.

(iv) It takes a long time to obtain results from field exposure tests unless the property changes leading to upgradation are detectable at early stages in the exposure.

(v) Exposure conditions cannot be controlled and the intensities of weathering factors are seldom measured, particularly at the micro-environmental level. It is difficult, therefore, from field exposure tests to identify mechanisms of degradation and to isolate the effects of various degradation factors.

(vi) Accelerated testing is satisfactory provided the artificial environment does not induce forms of degradations which do not occur in the service environment.

A typical case of service life prediction[17] of concrete coating used to restrict ingress of chlorides based on laboratory tests is illustrated in the ***Appendix.***

4.4. Mathematical and Simulation Modelling

Models used to consider effect of degradation can be deterministic or stochastic. Deterministic models do not consider scatter of concerned values. The model yields only one value of degradation, performance or service life that is often the mean value. In some cases, however, deterministic models are formulated to give a specific fractile value instead of the mean. In many cases, however, the deterministic models are insufficient to evaluate the risk of not reaching a definite figure of service life. In structural designs, therefore, stochastic approach is considered essential as the scatter due to degradation can be wide and the degree of risk may be high. In stochastic modelling for assessing remaining life, certain minimum reliability or failure probability is assumed. In stochastic design, deterministic design models are normally used for evaluating the mean degradation. To evaluate the standard deviation, a constant coefficient of variation is given:

Thus,

$$\sigma = v.\mu$$

Where,

σ = the standard deviation of degradation
μ = the mean of degradation, and
v = the coefficient of variation

Standard deviation can also be obtained analytically as follows:

$$\sigma^2(t_L) = \sum_{i=1}^{n} \left\{ \frac{\partial t_L}{\partial x_i} \sigma(x_i) \right\}^2$$

Where,

$\sigma(t_L)$ = the standard deviation of service life distribution (t_L)
$\sigma(x_i)$ = the standard deviation of parameter x_i
X_i = one of the n parameters in the service life model (t_L)

An example to calculate service life is illustrated in the ***Appendix.***

In the exercise for life prediction, one not only considers the target service life but also assumes the value of maximum allowable probability of not reaching the target service life. It is called the probability of failure. Hence, the probability of failure can be defined as the probability of exceeding or falling below a certain limit state, which may be an ultimate limit state or a serviceability limit state.

4.4.1. **Discrete time markov chain process** : Markov chain $\{X_n : n\ n \geq 0, 1, 2 \ldots\}$ is a discrete time stochastic process, which is used to model the deterioration process as a decay of condition ratings over time. A stochastic process is an indexed set of variables which evolve randomly over time. Stochastic methods rely on the analysis of performance data without consideration of the mechanisms involved. Here X_n is the bridge condition-state at the n^{th} time point.

The method simulates a natural deterioration process starting with perfect condition and proceeding with gradual and random degradation. However, unlike other probability methods, no parameters or assumptions for the type of distribution are needed. The state space S in the present case consists of condition ratings of the bridge components at any time described by a number from a pre-selected discrete numerical scale (say 1 to 5 with 1 and 5 being the initial and the worst states respectively).

In this process, the probability that an element transitions from condition state i to another condition state j does not depend how the element arrived at the i^{th} state. Probability of transition from state i to state j at time point n is denoted by $P_{ij}(n)$

Expressed analytically $P_{ij}(n) = P\{X_{n+1} = j \mid X_n = i\}$. These transition probabilities are given in a transition probability matrix [P] which is expressed as

$$[P] = \begin{bmatrix} P_{11} & P_{12} & \ldots & P_{1.M-1} & P_{1.M} \\ P_{21} & P_{22} & \ldots & P_{2.M-1} & P_{2.M} \\ P_{31} & P_{32} & \ldots & P_{3.M-1} & P_{3.M} \\ \vdots & \vdots & & & \\ P_{M1} & P_{M2} & \ldots & P_{MM-1} & P_{MM} \end{bmatrix}$$

Where,

$p_{ij} \geq 0$ for all i, j ε S,

$$\sum_{j \in s} p_{ij} = 1 \text{ for all } i \in S \text{ and } S = \{ 1, 2, \ldots\ldots M \}$$

M being the total number of degradation condition stages. Matrix [P] has rows and columns equal to assumed discrete condition, states (M).

Initially, it is necessary to assign p_{ij} values, e.g., 0.1, 0.2, 0.7 depending upon the probability of the element transitioning from its initial state, say 1 to 2, 3 or will continue to remain in 1 itself.

Existing bridge deterioration models used in the Markov chain theory invariably assume that

(i) the condition of structure cannot be improved during the process, and
(ii) the condition can either remain same or shift to the next state within one transition of 1 year step.

With the first assumption all transition probabilities below the diagonal probabilities are zero, since, e.g., in p_{21} condition state 2 cannot transit backwards to condition state 1, i.e., from existing state to improved state. Due to the second assumption, all probabilities above those next to diagonal ones are zero. As the structure must remain at the same state or drop to the next one only within one year step and cannot jump to any other lower state, sum of probabilities of remaining (diagonal elements) and dropping to the next state (elements next to the diagonal ones) must be 1. Thus, the probabilities of dropping to the next state can be calculated by subtracting the diagonal probability values from 1. Consequently, only the diagonal probabilities of the matrices are unknown parameters.

Thus, the transition matrix [P] can be expressed as

$$[P] = \begin{bmatrix} P_1 & 1-P_1 & \dots & 0 & 0 \\ 0 & P_2 & \dots & 0 & 0 \\ 0 & 0 & \dots & P_{M-1} & 1-P_{M-1} \\ 0 & 0 & \dots & 0 & 0 \end{bmatrix}$$

Where, p_i, i = 1 to M-1 represents the probability of remaining in the i^{th} state in the next transition. As M is the worst state in a state space of 1, 2,.....M and cannot further deteriorate, P_M will have to be equal to one. p_1 to p_M have numerical values between 0 and 1.

The expected condition of the bridge at a future time n or conversely, the expected time to reach any specific future state, can be calculated by using the following relationship:

$$|\pi^{(n)}| = |\pi^{(o)}| : [P]^n$$

Where, $\pi^{(n)}$ is the state probability vector at any time n consisting of the probability mass function (pmf) of X_n or the degradation index distribution vector for n^{th} year. $\pi^{(o)}$ is the initial state probability vector and $[P]^n$ is the n^{th} power of the transition probability matrix [P]. Vector matrix $[\pi]$ has elements equal to the assumed condition states (M). Thus, changes after n years can be predicted by multiplying the initial degradation index distribution $\pi^{(o)}$ by the transition matrix n times. The process is depicted in Fig. 14. If we assume structure initially in best condition then $\pi^{(o)}$ would be [1 0 0 ...0] meaning that structure is at degradation index indicating the best stage.

The Markov chain theory assumes that transition probabilities depend upon the current state irrespective of age of the bridge, thus only one transition matrix [P] is used for the whole life span. As a modification, some BMS softwares use

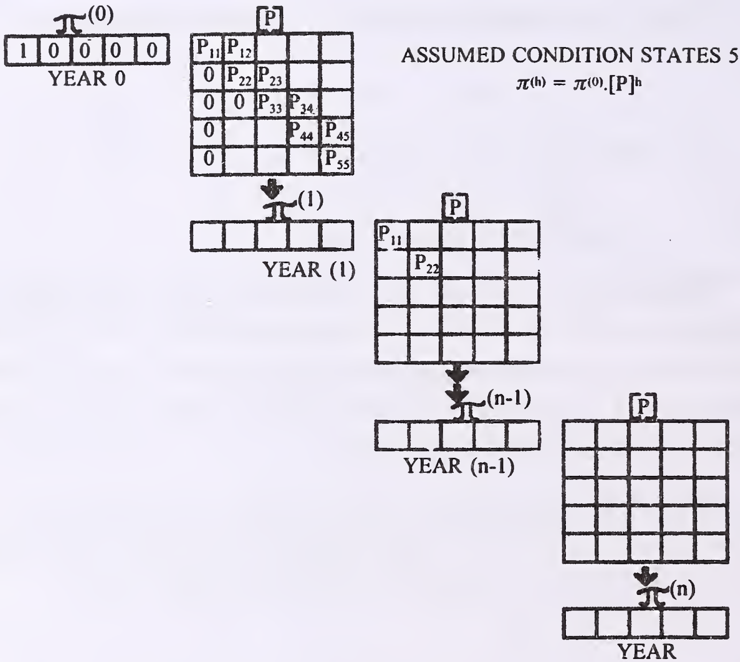

Fig. 14. Typical markov chain process[44]

different transition matrices for each age group, which means the stochastic nature of the deterioration process would depend upon both the current states as well as the ages of the bridges.

4.4.1.1. **Estimation of transition probabilities** : Two methods - 'Frequency approach' and 'Regression approach' have been suggested[19] to estimate transition probabilities.

In the Frequency approach, P_{ij} is calculated as

$$p_{ij} = \frac{n_{ij}}{n_i} \qquad i, j = 1, 2, 3..., M$$

Where, n_{ij} is the number of bridges originally in state i which have moved to state j in one step, and n_i, is the total number of bridges in state i before the transition. This approach would require at least two sets of inspection data pertaining to two different points in time.

In Regression approach, only one set of bridge data is needed. A regression function is first obtained by regressing condition ratings on ages. Transition probabilities are then estimated by fitting the regression function with the transition matrix.

Here degradation index distribution for each year $\pi^{(t)}$ is obtained by multiplying the degradation index distribution of the previous year by the transition matrix [P].

$$\therefore\ |\pi^{(t)}| = |\pi^{(t-1)}| \times [P]$$

The mean of the degradation index distribution at each year E(t, P) is obtained by multiplying the scale index vector R= | 0,1,2,3 .. M | by the degradation index distribution vector (Vector multiplication)

$$\therefore\ E(t, P) = |\pi^{(t)}| \times |R|$$

This mean of the degradation E is compared with the available reference degradation curve for that particular year. The probabilities p_i in the transition matrix are selected by minimising the sum of yearly deviations between the reference degradation curve and the Markov estimation for the degradation curve, i.e.,

$$\min SUMD = \sum_{t=1}^{N} | s(t) - E(t, P) |$$

Where,

SUMD	=	the sum of deviations at each year
N	=	the number of years within service life
P	=	the transition matrix with unknown probability elements p_i
s(t)	=	the value of the degradation index from the reference degradation curve at year t
E (t, P)	=	the mean of the degradation index distribution calculated by the Markov chain method at year t.

It has been observed that the tractability of the Markov chain mean curve is best with respect to square root pattern of reference degradation curve.

4.4.1.2. **Problems in application :** In the absence of any other more accurate method, Discrete Time Markov Chain (DTMC) has been gaining entry into the Bridge Management System software packages. A network optimisation system for bridge improvement and maintenance PONTIS of FHWA assumes transition probabilities to depend upon the current states and not on the ages of the bridges and hence uses only one transition matrix for the whole life span[19]. BRIDGIT is yet another software developed in the U.S.[27]. There are, however, several issues[19] connected with this method which need to be examined by applying the technique to practical problems:

(i) Suitability of Markov Chain theory in bridge deterioration modelling

This can be confirmed only from the accuracy achieved in prediction of remaining life. Presently, structural reliability based methods are the only alternative option available. Confirmation from the actual performance is a slow process, till such time it is necessary to keep applying the DTMC on a variety of bridges with whatever initial data is available.

(ii) Suitability of condition rating as the bridge performance index

Although, it is relatively simpler it has, however, been observed that condition rating in numeric scale is not adequate as performance index. Condition rating does not reflect the structural integrity of the bridge. It has been suggested that bridge deterioration modelling should include load rating.

(iii) Method of estimating the transition probabilities p_{ij}

This difficulty is expected in the beginning as no field data are available. However, based on instrumentation, non-destructive testing and visual inspection an experienced bridge engineer can assign number representing condition state after each inspection. With the availability of condition states from the past inspections, the transition probability matrix can be further improved.

Readers are advised to refer the modern text books on probabilistic methods for detailed treatment of the method[38, 42, 44].

4.4.2. **Reliability based methods :** Decisions to continue service and/or to perform maintenance need to be supported by quantitative evidence that the strength is sufficient to withstand future extreme events within the proposed service period with an acceptable level of reliability. Structural reliability methods provide the basis for this evaluation. Statistical simulation is used in stochastic methods to estimate the likelihood of a bridge component being in a certain state of condition at some point of time in future. In contrast to stochastic methods, reliability based methods do require that the mechanism of degradation is understood. As the service life is often represented by a failure distribution, and since the variable conditions exist in both the loading and resistance characteristics of any bridge, it is claimed that reliability theory can be an effective modelling tool. It involves a systematic probabilistic procedure for quantitative prediction of service lives of materials and components. Main features of this approach are given below:

Structural loads, engineering material properties and strength degradation mechanisms are random function of time, and, therefore, the performance of the bridge is also of random variable nature depending upon material characteristics, workmanship, maintenance standard, etc. The margin of safety M(t) at any time t can be expressed as:

$M(t) = R(t) - S(t)$, in which $R(t)$ and $S(t)$ are resistance and structural actions respectively. The approach is based on the criterion that the probability of the resistance of the structure being smaller than the load within the target service life (t_g) is smaller than a certain maximum acceptable failure probability (P_{fmax}).

Mathematically expressed,

$$P\{failure\}_{tg} = P\{R-S<0\}_{tg} < P_{fmax}$$

Where, P $\{failure\}_{tg}$ = the probability of failure of the structure within tg. The problem can be solved if the distribution of the load and the resistance are known and the P_{fmax} is defined.

If R (τ) and S(τ) are instantaneous physical values of resistance and the load at the moment τ, the failure probability P_f in a life time t can be expressed as:

$$P_f(t) = P\{R(\tau) < S(\tau)\} \qquad \text{for all } \tau \leq t$$

As R and S are stochastic quantities with time-dependent or constant density distributions, $P_f(t)$ can be expressed as:

$$P\{R(t)< S(t)\}$$

It will be seen from Fig. 15 that the failure probability increases with time. At t = 0, the density distributions of load and resistance are far apart and the failure probability is small. With time, the distributions approach each other, forming an overlapping area of increasing size. This overlapping area illustrates the failure probability.

Loads (S) and resistance (R) are generally assumed as normally distributed. Considering continuous distributions, the failure probability P_f at certain moment of time τ can be determined using the following convolution integral:

$$P_f(t) = \int_0^{\infty} F_R(s) f_s(s) ds$$

Where,

$F_R(s)$ = the distribution function of R

$f_S(s)$ = the probability density function of S, and,

s = the common quantity or measure of R and S

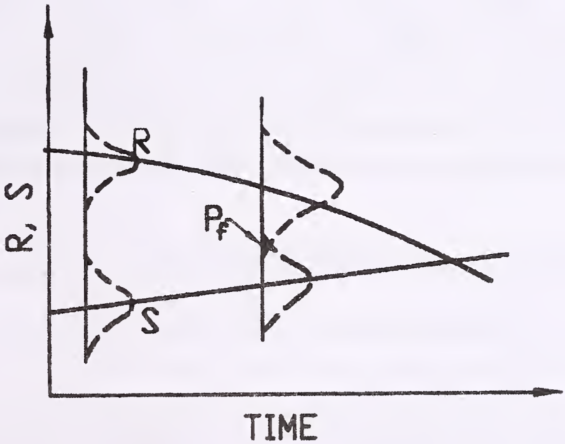

Fig. 15. The increase of failure probability (illustrative presentation[44])

In this case, the failure probability can be determined using the test index ß

Where,

$$\beta = \frac{\mu[R,t] - \mu[S,t]}{(\sigma^2[R,t] - \sigma^2[S,t]^{\frac{1}{2}}}$$

Here μ and σ denote the mean and standard deviation respectively. In structural design, the index ß is referred to as safety index or reliability index. Very often R or S is constant in which case β

$$\beta = \frac{r - \mu[S,t]}{\sigma[S,t]} \quad \text{(for R constant)}$$

The failure probabilities corresponding to β are available as tables. To obtain the distribution of service, the failure probabilities have to be solved with several values of t (e.g., 10, 20, etc. years).

As an example the case of carbonation of concrete is considered[44]. Failure is assumed to occur when carbonation depth exceeds the depth of reinforcement. Here S is the time related to carbonation process and R the constant concrete cover is also considered as stochastic quantity. For the numerical data given in section 3.1, the test index β can be expressed as

$$\beta = \frac{25-1800(30+8)^{-1.7}\,t^{\frac{1}{2}}}{\left[-\left(0.6\times1800(30+8)^{-1.7}\,t^{\frac{1}{2}}\right)^{2}+(0.2\times25)^{2}\right]^{\frac{1}{2}}}$$

The corresponding probability distribution function and probability density function are given in Figs. 5 and 6.

For service life prediction and reliability assessment, one is more interested in the satisfactory performance over some period of time, say (0,t). The probability that the structure would survive during interval (0,t) is defined by reliability function L (0,t), in other words, it is the probability that the time to reach failure exceeds the time t, i.e.,

$$L(t) = P[T>t]$$

where,

$$t \geq 0$$

Where, T is the non-negative random variable representing the time to failure. For detailed understanding of the procedure, attention is drawn to specialised literature[19,38,44,45].

4.4.3. **Fracture mechanics approach**[11]**:** Techniques based on fracture mechanics have also been used in modelling creep, fatigue or fracture degradation of metals. As an alternative to cumulative damage assessment based on S-N curves, fracture mechanics approach based on cumulative crack increments has also been attempted. Flaw size is an additional variable in this approach, and fracture toughness replaces strength as the relevant material property. This approach appears to be more useful in fatigue related studies, since the measure of damage, i.e., the crack length can be physically assessed whereas S-N curve approach does not allow for direct measurement of damage and damage increments.

5. GENERAL PROCEDURE FOR LIFE ASSESSMENT

Procedures are available in the literature to predict the service life in the case of building materials and components[12]. In fact there cannot be one general procedure for all types of bridges and environments. Considering the history and the environmental conditions of the bridge in question, the procedure will require modification. The deterioration and actual damage levels differ from structure to structure, making it impossible to lay down a strict framework of the procedure.

Before predicting the remaining life, it is essential to fix performance criteria for the bridges and in the case of reliability based approach, the acceptable level of probability of failure. Factors that would affect the safety and serviceability of the bridge, inspite of periodic routine maintenance, are required to estimate the remaining life. A concrete bridge component reaches the end of its functional service life (Fig. 1) when the level of physical damage warrants not just repair but rehabilitation of the component[15]. There are several examples of wear and tear affecting the riding quality, or of malfunctioning of bearings, expansion joints, etc. These are the results of lack of maintenance and cannot be the governing considerations to determine the remaining life.

5.1. **Broad Steps**

Based on the various aspects discussed in the earlier sections, a comprehensive procedure to assess the remaining life of an existing bridge is summarised below. All the listed activities may not be necessary in all the situations. However, for the sake of comprehensiveness, they are listed together.

Step 1: Defining of minimum acceptable performance limits, e.g., span/deflection ratio, deflection recovery under loads, sag, crack widths, riding quality, vibration levels, acceptable probability of failure, etc.

Step 2: Characterisation of existing status of the bridge: Status of constituting materials, component, etc. based on condition survey, i.e., visual inspection for cracks, chloride content, carbonation level, porosity, corrosion mapping, hammer tapping, cover scan, strength related properties through hammer, UPV and core tests, petrography, radiography, endoscopy, etc., if situation demands, condition state rating (component wise), installation of strain, rotation, deflection and temperature sensors (for periodic evaluation) followed by static load test and dynamic signature tests.

Step 3: Analysis of the environment (Macro, meso and micro level)

- Chloride ion contents from concrete samples
- Quality of water below the bridge
- Daily and seasonal temperature changes at the site, within the box (in the case of closed sections), and within the concrete
- Relative humidity level
- Air pollution data
- Live load induced stress histograms
- Water discharge, scour level
- Drainage arrangement at the deck level

Step 4: Identification of critical degradation causing agents and their rates, e.g., Carbonation induced corrosion, Chloride ingress-induced corrosion, Live load induced fatigue, wear-tear, abrasion, scour.

To determine deterioration rate ideally one would require past periodic data on the performance and access to the accelerated testing facility. But this would rarely be available. In the absence of the same, the identification of potential degradation agents and their rates will largely depend upon investigator's engineering judgement and past experience. While selecting the degradation rate, it should be kept into consideration that some prestressed concrete bridges have suffered premature deterioration[52,53]. Guess work will have to be made based on the data available from the in-situ tests, quantification of deterioration vs. time rates based on field data from similar cases, empirical formulae, e.g., on corrosion, carbonation, chloride diffusion discussed in section 3.0, results from accelerated tests, etc.

Maintenance of bridges should be need based addressing the causes of damage.

Influence of repairs or the level of maintenance measures to be undertaken in future may also have bearing on the rate of deterioration. However, for conservative estimate, this may either be ignored or suitable allowance be given.

Step 5: Identification of structural failure modes : Scanning of critical sections/zones based on Design calculations, Construction data, Condition survey, Observation during load test, Maintenance record, etc.

Conversion of deterioration processes into structural failure modes, and estimation of critical structural parameters, such as, Loss of prestress, Loss of bond and area of steel, Loss of composite action, Change in stiffness- E value, Excessive geometrical changes, e.g., settlement, sag, etc.

Step 6: Prediction of remaining life based on most critical deterioration rate and corresponding structural failure mode.

No unique procedure is feasible for all types of situations. Available approaches include,

- Estimation based on engineering judgement
- Regression function to model deterioration based on performance of bridges having similar materials in similar exposure conditions.
- Probabilistic approaches, such as, Markov chain process, structural reliability approach.

As the entire procedure becomes qualitative based on several assumptions, it is desirable to supplement the prediction with more than one approaches.

5.2. Limitations

It is to be appreciated that the calculations of remaining life involves many uncertainties, e.g.,

- range and accuracy of in-situ investigation procedures
- estimated deterioration rates
- selection of critical damage mode which would become the cause of failure in future
- assumption of environmental conditions and the level of repair/ maintenance during the remaining life, since the long term performance of the bridges would depend upon their periodic maintenance.

In all above considerations, sound engineering judgement and past experience of the investigator plays a crucial role. The service life determination helps to forecast the time frame for possible repair or rehabilitation in future. Therefore, from practical point of view, even a broad range of time period in years would be useful.

The prediction of residual service life, where long periods (e.g., 50 - 100 years) are involved or where no visible signs of distress are apparent, can at times be difficult. But in such cases, the situation also allows more inspections to apply mid-course corrections. From the present status of the knowledge and technology, it appears possible to carry out a qualified service life prediction. But in damaged structures, there will always be a need for more intensive inspections during the predicted period, no matter how advanced the service life modelling is.

6. ACTION PLAN

The status of life assessment of bridges world wide is at such a stage that while its importance is being recognised, scientific data on material degradation and bridge performance is lacking. As a consequence, the available analytical tools for life prediction have remained untested. Therefore, in order to create awareness to monitor bridge performance during its service life and to develop data bank several steps need to be initiated in Highway Departments and Research Institutions.

It is necessary to characterise degradation agents, understand damage modes, develop accelerated test procedures and generate data, document field performances, report bridges in distress to the concerned authorities, test the available and newly emerging mathematical models, and above all an institutionalised arrangement to overview and co-ordinate these tasks on a long term basis. The ultimate objective is to introduce achievements of material research of concrete structures into the art of structural design involving 'time' dimension. Specific tasks requiring actions are the following:

(a) To develop effective mechanism for reporting data on actual in-service performance of bridges including lessons from cases where failures have clearly occurred.

(b) Identification and grouping of bridges based on bridge geometry and exposure to aggressive environment (e.g., aggressive climate, heavy traffic, scour, etc.)

(c) Guidelines for minimum expected performance levels.

Structurally acceptable limits, such as, on deflection, crack widths, riding quality, vibration level prescribed in the codes of practice, provide one set of threshold values. However, the limiting values indicating the end of the functional service life have bearing on techno-economic considerations. A bridge component reaches the end of its functional life when the level of damage warrants not just repair but rehabilitation of the component. Unlike the end of the structural service life, which often can be objectively defined on the basis of readily observable distress signs, the end of functional service life, is ultimately a matter of opinion.

(d) To develop data bank for documentation of periodically collected data from bridges and arrangement for drawing of statistical conclusions.

(This feedback would form the source and the data base required to assign transition probabilities in Markov chain process as well as to develop regression expressions connecting condition ratings with the potential degradation agents.)

(e) Collection of data on the environmental factors causing degradation and improved methods to measure the intensities of the factors.

(f) To develop improved knowledge of the mechanisms by which constituting materials (including coatings/sealers) degrade.

This would involve laboratory tests under controlled conditions to collect data on deterioration rates, such as, in corrosion-affected passive and prestressed steel, rate of carbonation in concrete, fatigue accumulation under programmable loading. Deterioration rates referred in earlier sections relate to specific case studies. The scope would increase as more and more forms of deterioration processes are identified. This would also lead to establishing laboratory infrastructure for testing, e.g., weathering chambers, programmable pulsating loading facility, etc.

This should also cover identification of degradation phenomena acting concurrently and development of methods to simulate or account for synergism amongst degradation factors. This is essential since it has been the experience that failure - mild or severe - generally will be due to combination of factors, many of which are not amenable to mathematical modelling.

(g) To develop instrumentation based performance monitoring procedures incorporating time saving, easy to handle improved tools and methods for measuring degradation in-situ and a mobile laboratory operated by trained engineers to evaluate the condition of the structure on site during testing.

(h) Testing efficacy of different service models

Research programme to analyze concrete structures subjected to time-related degradation processes need to be planned. Not only are 'first level' cross-sectional studies but 'second level' related to the behaviour of the component and 'third level' related to the response of the whole structural assembly also need to be included in such a programme. It is necessary to understand accuracy of different life prediction methods, as well as the difficulties in their applications to field examples. This is true particularly in the cases of Discrete Time Markov Chain (including its procedure to evolve elements of Transition Probability Matrix) and structural reliability methods. Application of these techniques to a large number of existing and new bridges where in-situ test data on a periodic basis is available, needs to be undertaken.

(i) Development of durability design philosophy

Control of the durability is gaining increasing importance in the design of concrete bridges. It is necessary to introduce into conventional structural design, structural concept of targeted service life, general theory of structural reliability and available calculation models for the commonest degradation processes. Concept of durability at the design stage would enable fixing durability related parameters, such as, concrete cover, properties of materials, amount of reinforcement and dimension of the members taking into account the actual degradation process. Further the introduction of 'time' element in the design, availability of periodically updated degradation rates during service life and the predefined targeted service life, would all help to plan the scope and schedule of maintenance measures.

It has to be appreciated that estimation of remaining life is not a straight-jacket exercise but is an inter-disciplinary science covering as divergent subjects as material science, electro-chemistry, fracture mechanics, fatigue, statistics, probability theories, instrumentation, besides failure mechanisms of structural engineering. In an effort to expose the readers to the recent developments in all the related disciplines, it is likely that the document would give the impression of being less user-friendly and wanting in precise steps to be followed by the prospective user in handling the practical cases.

However, it will be realised that the life prediction is one of today's front line areas of research and given the complexities involved, every investigation has to be bridge-specific. While it was necessary to touch upon various related subjects, being an approach document, any attempt towards spelling out precise step-by-step procedure would have been inconsistent with the level of development of the subject. Need in fact is to draw up a long term plan and co-ordinated efforts from bridge owners, designer, academic and research establishments, so that at least the qualitative estimation of health and the remaining life enters the Bridge Maintenance Systems in future. The purpose of this document would be served if it triggers action in this direction.

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Appendix

NUMERICAL ILLUSTRATIONS FROM LITERATURE

Being a newly emerging area of R&D, there are not many reported case studies of practical applications. Three numerical illustrations from the literature are, however, presented in the following, primarily to bring out the status of the subject. They cover the following cases:

- RC column and beam under degradation of both concrete and steel[44].
- Bridges with piers under chloride attack using in-situ tests[39].
- Concrete coatings used to restrict ingress of chlorides, using accelerated tests[17].

I. Service Life Predictions in RC Column and Beam[44]

Cross-sections of a simple axially loaded square column and a beam are considered to illustrate life estimation under degradation in both concrete and steel (Figs. 16 and 17).

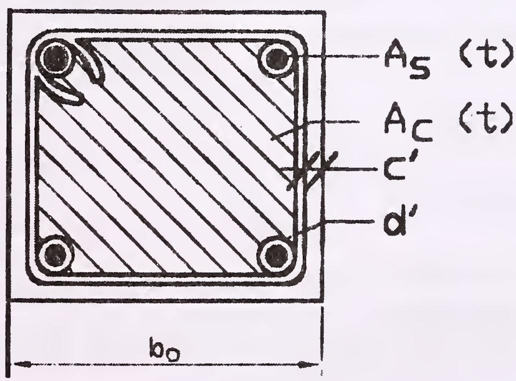

Fig. 16. Cross-section of a column[44] (degradation both in concrete and steel)

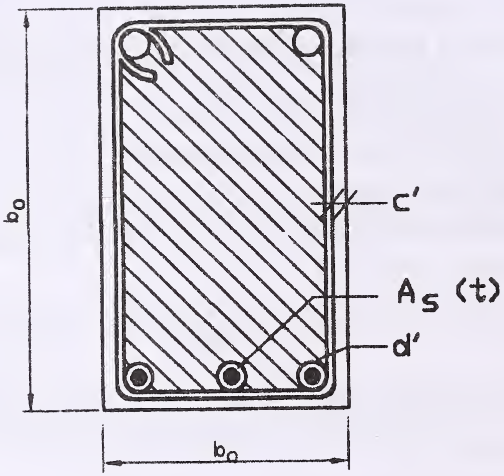

Fig. 17. Cross-section of a beam[44]

RC Column

Load bearing capacity

$$R_d = A_c(t)\,\frac{f_{ck}}{\gamma_c} + A_s(t)\,\frac{f_y}{\gamma_s}$$

Where, $A_c(t)$ and $A_s(t)$ are areas of cross-section of concrete and steel respectively as function of time t, and hence can be expressed as:

$$A_c(t) = (b_o - 2c'(t))^2$$

$$A_s(t) = 4\frac{\pi}{4}(D_o - 2d'(t))^2$$

Where c' = degradation model of concrete expressing the depth of deterioration of concrete and

d' = degradation model of steel expressing the depth of corrosion in reinforcement

b_o and D_o are the initial width of column and diameter of steel respectively.

Load bearing capacity at any time t $R_d(t)$ can be expressed as:

$$R_d(t) = R_{do} - 4\left[b_o C'(t)\frac{f_{ck}}{\gamma_c} + \pi\, D_o d'(t)\frac{f_y}{\gamma_s}\right]$$

Where, R_{do} is the initial capacity.

Depending upon whether c'(t) and d'(t) are linear or accelerating/retarding models, the load bearing capacity will also show similar trend. Fig. 18 shows the reduction in A_c, A_s and R_d expressed in per cent for a typical axially loaded column when c' and d' are assumed to be linear with time. Calculations have been done for

b_o	=	500 mm
D_o	=	25 mm
f_{ck}	=	40 MPa
f_{yk}	=	400 MPa
γ_c	=	1.5
γ_s	=	1.15
c'(t)	=	0.4 mm/year
d'(t)	=	0.04 mm/year

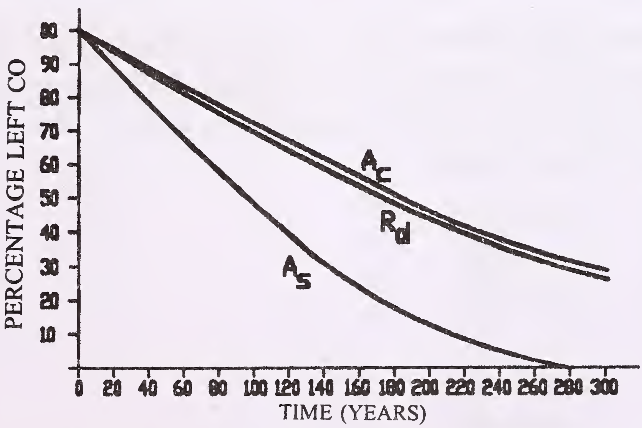

Fig. 18. Reductions in material cross-sections and compressive capacity of a column[44]

RC beam

Load bearing capacity of beam is similarly determined by calculating the moment of resistance. For more details see Ref.[44].

Fig. 19 shows the reduction in A_s, z, R_{ds} and R_{dc} expressed in per cent for a typical beam with following dimensions and

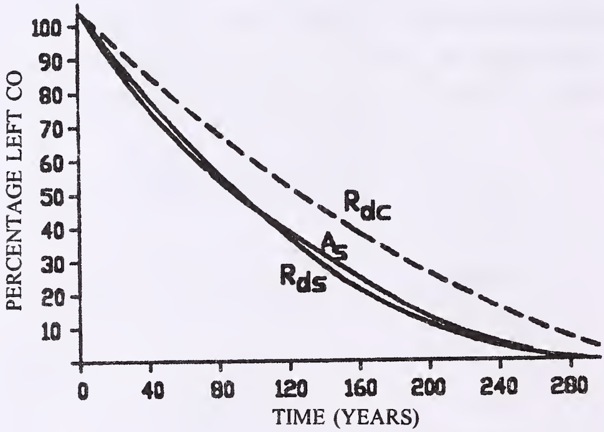

Fig. 19. Reduction in the bending capacity of a beam[44]

with constant degradation rate c' and d'. Here R_{ds} and R_{dc} are the moments of resistance corresponding to under-reinforced and over-reinforced section, respectively, and z is the internal lever arm of the moment

b_o = 400 mm
d_o = 700 mm
D_o = 25 mm
N_s = 3
f_c = 40 MPa
f_y = 400 MPa
E_s = 200000 MPa

$$E_c = 9500(f_{ck} + 8)^{\frac{1}{3}} \text{ N/mm}^2$$
$$\gamma_c = 1.5$$
$$\gamma_s = 1.15$$

II. Service Life Calculation in the Case of Chloride Attack[39]

The Danish Bridge Maintenance System comprises modules where service life calculations are necessary for the optimum use of the system. To meet the requirement of the Danish Road Directorate, Henriksen of RH&H Consult has reported a test-cum-service life calculation methodology using services of a mobile laboratory to evaluate the condition of the structure on-site during testing. The test results are used to determine the actual position of the structure on the service life deterioration *vs.* age curve and obtain the four points T_2, T_3, T_4 and T_5 on the time scale (Fig. 2). The table below shows the examples of service life calculations in the case of chloride attack on the piers of 5 different bridges. The initiation time

Bridge No.	Cs (%) Ground level	D (mm²/y) Ground level	Time T_2*	T_3*	T_4*	T_5*
70.0146	0.08	19	∞	∞	∞	∞
70.0182	0.07	110	2000	2010	2015	2025
70.0186	0.13	36	1988	1988	2003	2018
70.0036	0.08	79	1989	1999	2004	2014
70.0047	0.06	16	∞	∞	∞	∞

* with critical chloride level at 0.05 per cent of mass of dry concrete weight.

Cs - Surface chloride coefficient
D - Diffusion coefficient
1993 - Time of investigation

(T_2) is calculated using FICK's 2nd Law on the basis of chloride measurements carried out on the structure. The results of chloride measurements are used for fitting the curve to FICK's 2nd Law (Fig. 20).

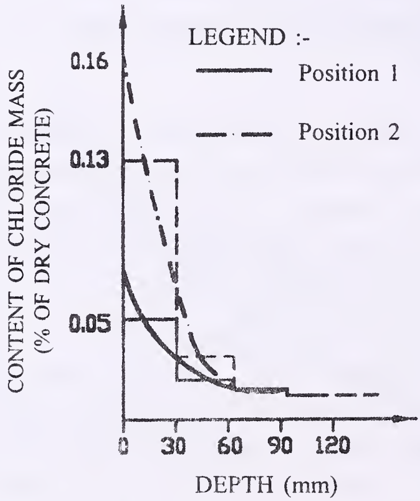

Fig. 20. Chloride measurements on a bridge deck

The rate of deterioration of structure used for evaluating points T_3, T_4 and T_5 is estimated on the basis of measurement on the structure of the electrical resistance, the moisture content, the porosity of the concrete and the annual average temperature and moisture variation. These are the primary parameters for calculating the deterioration of concrete.

The time periods are intended to enable plan the bridge repairing priorities aimed at optimising available maintenance funds. For more details see Ref. 39.

III. Life Prediction of Concrete Coating Based on Accelerated Tests[17]

Service life prediction of concrete coating used to restrict ingress of chlorides from salt-water environment based on accelerated tests in the laboratory is described in the following. The example illustrates the application of accelerated test as well as simulation of field condition.

Series of tests are performed in the laboratory on specially designed surface coated concrete specimens immersed in sodium chloride for a long period (one year in the present case).

Due to the presence of coating, surface chloride concentration is not constant. The equilibrium chloride concentration (kg/m^3) assumed 1.3 cm below the surface is hence treated as surface concentration. The regression analysis carried out on the test result values of concentration at 1.3 cm from the surface shows that it varies as square root of time, (=k $t^{1/2}$) where k is the chloride ingress rate through coated or uncoated layer.

The solution to the problem of semi-infinite medium whose surface concentration varies in proportion to the function of time (square root) is obtained by Laplace Transform of diffusion equation, as:

$$C(x,\ t) = k\sqrt{t}\left[e^{\frac{x^2}{4}.D_c.t} - \frac{x\sqrt{\pi}}{2\sqrt{D_c t}}\left(1 - \frac{\text{erf}(x)}{2\sqrt{D_c t}}\right)\right]$$

Where,

k = coating characteristic constant

t = time

x = depth

C = Chloride concentration

D_c = diffusion constant

The tests results generate data to know the 'k' values for the coated as well as the uncoated surfaces. Since these values are valid for the laboratory scale tests, they have to be modified for their actual application in the field. Hence,

$$K_{modified} = \frac{K_{coating}}{K_{concrete}} \times K_{field}$$

Here $K_{coating}$ and $K_{concrete}$ denote the regression based values for K coefficients obtained in the tests on coated and uncoated specimens. K_{field} is the value calculated from the field data.

The following data is considered:

$C(x,t)$	=	0.71 kg/m³ (chloride threshold value)
x	=	4.1 cm (location of steel)
D_c	=	0.84 cm²/year
t	=	50 years (desired corrosion protection period)
$C_{surface}$	=	8.9 kg/m³ (Surface chloride concentration related to severe condition)
$K_{coating}$	=	1.928
$K_{concrete}$	=	7.829 (obtained on regressing the test data)

Using above equation and above data with all quantities except 't' being known, the protection time provided by the specific coating can be calculated. [In the present case t equalled 29 years Refer [17, 18, 47] for more details.