A New Approach to Durability Design Using Risk Analysis

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Abstract

Current codes use a deemed-to-satisfy approach to durability design. This approach has severe limitations, principally in failing to offer a definition for design life and what constitutes its end. It also fails to acknowledge the fact that, in practice, deterioration is progressive. An alternative approach to durability design is proposed which uses probabilistic analysis. This provides a basis both for a rational definition of design life in relation to the risk of exceeding defined serviceability limit states and for quantifying the cost-effectiveness of different measures for enhancing durability. The output from the probabilistic analysis also provides a rational basis for Life Cycle Costing.

The paper describes the proposed probabilistic approach to durability design, gives an example of its application to chloride induced corrosion and shows how the results can be applied to Life Cycle Costing.

Introduction

Durability design is currently based on a deemed-to-satisfy approach. Limits are given for w/c, strength grade, cement content and cover, and if these requirements are met, the structure is deemed 'durable'. If the observed occurrence of premature deterioration was low, then this approach would have to be considered acceptable. However, corrosion of reinforcement continues to represent the single largest cause of deterioration of r.c. structures worldwide. The problem is variously attributed to inadequate specification, poor design detailing and construction defects, such as poor compaction or curing or low cover. However, field data suggests that, in the most aggressive exposure conditions, there is an unacceptably high risk of premature deterioration even when some code requirements are met(1). This is not to say that all structures designed using these codes will deteriorate prematurely, only that the extent of premature deterioration and the associated cost of repairs will continue to be unacceptably high.

In addition to failing to keep deterioration to an acceptably low level, the deemed-to-satisfy approach is limited in several other major respects. Principally it fails to acknowledge that structures deteriorate progressively. When codes do declare a design life (and many fail to do this) then it is assumed that, if the specification is met, then the life will be achieved - but clearly a structure with a 50 year design life will not suddenly deteriorate to a state of unserviceability after 50 years. Often deterioration will commence before the design life is reached, hence the healthy growth in the concrete repairs market. Another major deficiency in the current approach to design is the failure to define what constitutes the end of the design life. What is the serviceability limit state which must be exceeded before the structure has "failed" with respect to durability?

These deficiencies in the current approach to durability design reflect the difficulties and uncertainties in predicting the long term performance of structures. For example, an industry survey carried out in the UK as part of a review of BS 7543, Guide to Durability of Buildings and Building Elements, Products and Components, indicated that it was rarely used in design because of lack of reliable predictive models and performance data [2] and designers were unwilling to accept responsibility for long term performance.

In structural design, however, these uncertainties are dealt with probabilistically, although this is rarely declared explicitly within codes. It is implicit, however, in those codes which define the risk of failure using a reliability index.

A probabilistic approach to durability design is proposed [3] which is similar to that used in structural design. This requires acceptance of the fact that variability, similar to that for mechanical behaviour, also exists in relation to the properties of concrete which influence durability. Furthermore, the inherent variability of the exposure condition or 'environmental loading' on the structure must also be taken into account.

Approach to Probability Based Durability Design

The proposed approach for durability design is similar to that used in structural design[4,5]. In its simplest form this is presented as a limit state function of the form;-

$$R(t) - S(t) \geq 0 \qquad (1)$$

where R(t) is the resistance and S(t) is the load, and both are assumed to be time dependent. For structural design it is usual to assume that the strength remains constant and that the loads, even if fluctuating, can be characterised by a single value. In each case partial safety factors are applied to take account of variability and uncertainties, leading to the design values.

Durability is, by definition, time dependent and hence these simplifying assumptions cannot be made. Furthermore, there may be several serviceability limit states. Siemes and Rostam[5] have described two approaches to durability based on the 'intended service period design' and the 'lifetime design'. These are illustrated in Figure 1.

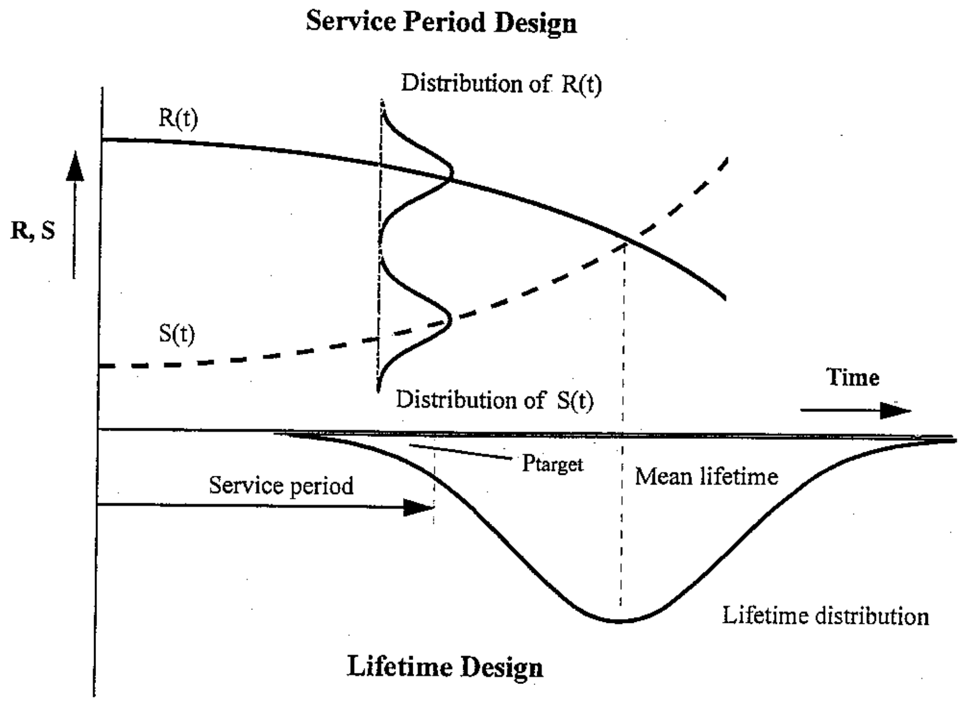

Figure 1 *Service period design and lifetime design*[4]

The reliability functions are as follows:

Intended service period design

$$P_{f,T} = P\{R(t) - S(t) < 0\}_T \leq P_{target} = \Phi(-\beta) \tag{2}$$

where $P_{f,T}$ is the probability of failure within the intended service period, T

P_{target} is the accepted maximum value of the probability of failure

Φ is the standard normal distribution function

β is the reliability index (normally given in codes instead of the failure probability) and is the number of Standard Deviations from the mean of a normal distribution outside which the area under the curve represents the probability of failure. It can be obtained from tables[6] and the relationship is as follows:-

$-\beta$	1.3	2.3	3.1	3.7	4.2	4.7
$P_{f,T}$	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}

Lifetime design

$$L = T\{R,S\} \quad \text{where L is the life of the structure} \tag{3}$$

and $$P_f = P\{L < T\} \leq P_{target} = \Phi(-\beta) \tag{4}$$

Each of these approaches uses the same information and will lead to the same result.

For structural design, the risk of failure must be very low. For example, for ultimate limit state (i.e. collapse), Eurocode 1 is based on a probability, P_{target}, = 7 x 10^{-5} (β = 3.8) for T = 50 years. A higher level of risk may be tolerated in relation to serviceability limit states as the consequences are much less severe. Furthermore, there is generally a visual warning long before the defect has serious safety implications, with the opportunity for intervention to reinstate the structure and to prevent further damage. There are cost implications, however, and for this reason the risk of corrosion should still be designed at an acceptably low level. A probability of the onset of corrosion of 10^{-2} ($-\beta = 2.3$) may be more appropriate.

Serviceability Limit States

A critical feature of durability design is the definition of serviceability limit states. For corrosion of reinforcement, the limit states illustrated in Figure 2 are proposed, depending on the nature and location of the structure and the criticality of the element, or part of the element, being considered.

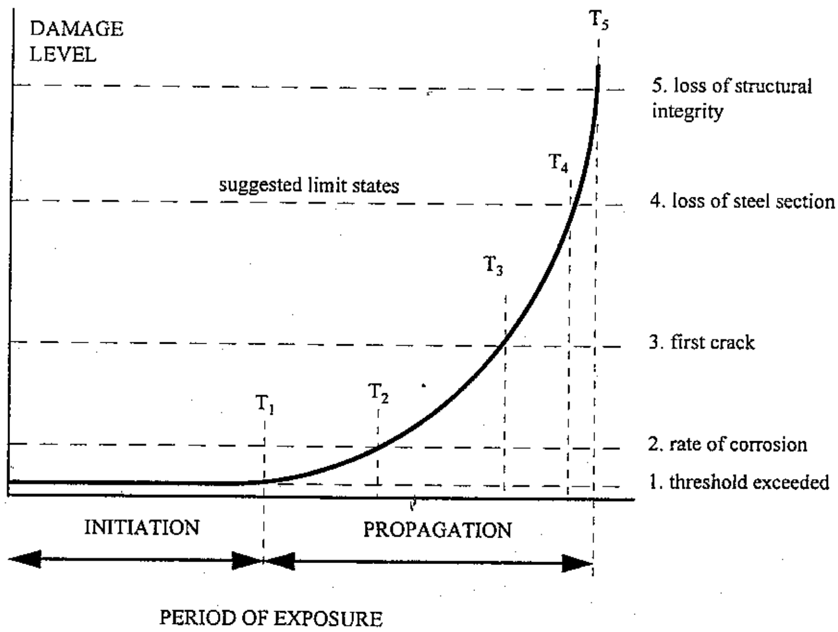

Figure 2 *The general deterioration model and suggested serviceability limit states.*

Onset of corrosion - the time to the onset of corrosion is defined as the Initiation Phase. Initiation is most commonly deemed to have occurred either when the threshold chloride level is reached or when the concrete has carbonated to the depth of reinforcement, but it can also be defined by the time to exceed a defined rate of corrosion. In conditions in which pitting can occur the engineer may decide that no corrosion is acceptable and hence specify the service life to be the onset of corrosion. Using this limit requires either the prediction of chloride ingress and comparison with a selected threshold value or the prediction of carbonation depth and comparison with cover. Functions defining the probability, P_{IT}, of Initiation after time T are, therefore, of the form;-

For chlorides, $$P_{I,T} = \{C_x(t) - C_{th}(t) < 0\}_T \leq 10^{-2} \qquad (5)$$

where C_x is the chloride level at the reinforcement
and C_{th} is the chloride threshold level for corrosion

For carbonation, $$P_{I,T} = \{X_c(t) - X_r < 0\}_T \leq 10^{-2} \qquad (6)$$

where X_c is the carbonation depth
and X_r is the cover to reinforcement

Time to first cracking - this is usually the time at which some intervention is made. Very little corrosion (less than 100 microns) is needed to cause cracking(7). Using this limit requires the prediction of the rate of carbonation or chloride ingress, the consequential rate of corrosion and the amount of corrosion required to cause damage. Alternatively, if there is sufficient experience with structures in a particular environment, a predetermined propagation period may be used. It must be appreciated, however, that in using this approach the designer is allowing corrosion to occur and relying on achieving a predicted period of propagation. If there is a risk of chloride induced pitting corrosion with rapid loss of steel section, or if the consequences of a spall may be catastrophic e.g. a small piece of concrete falling from a bridge through the windscreen of a car travelling at high speed or from a building onto a passing pedestrian, then caution must be exercised.

Specified loss of section - In remote structures it may be acceptable to design to allow a specified loss of steel section. This would only be appropriate, however, where the consequences of spalling are acceptable. In practice, except for temporary or non-critical structures in remote locations, it is not expected that this approach is likely to be adopted.

Mathematical Models

The probabilistic approach requires the prediction of service life and mathematical modelling is an essential feature of the methodology. Engineers are comfortable with the use of relatively complex equations to predicted structural behaviour but there is still some reticence to use a similar mathematical approach for durability design.

To design a structure with a quantifiable service life it is necessary to develop a mathematical model for the mechanism(s) of deterioration. For concrete, all of the deterioration mechanisms are complex interactions of physical and chemical processes. A scientifically based model to predict such complex and varying phenomena, even if achievable conceptually, would be extremely difficult to define mathematically and to execute quantitatively.

At the opposite end of the spectrum, simple empirical models based on a large number of observations may not be sufficiently flexible to deal with conditions outside the scope of the data used in developing the model. From an engineering point of view, it is desirable that models, whilst being sufficiently accurate, are relatively easy to use, relying on mathematical formulations which do not require complex methods of solution, and input data which can be obtained from laboratory or field tests that are relatively fast, easy and economical to perform. Such input data are also subject to uncertainty and it is not surprising, therefore, that the majority of models currently available for design purposes rely on a compromise being achieved, being scientifically based, but using simplifying assumptions to make them acceptable to practising engineers. At the very least a deterioration model must include terms which represent the environmental loading and the resistance offered by the concrete, and all terms must be measurable.

Model for Chloride Ingress into Concrete

As an example, for predicting the ingress of chlorides into concrete the following equation is proposed:

$$x = 2 \cdot C\sqrt{f_t \cdot D_{ca(m)} \cdot f_c \cdot f_e \cdot t \cdot \left(\frac{t_m}{t}\right)^n} \tag{7}$$

in which,

$$C = \mathrm{erf}^{-1}\left(1 - \frac{C_x}{C_{sa}}\right) \tag{8}$$

and:
- $D_{ca(m)}$ is the apparent diffusion coefficient measured at time t_m
- C_{sn} is the surface chloride level
- C_x is the level of chloride at depth x
- n is the age factor applied to D_{ca}
- f_t, f_c, f_e are constants which take account of the method of test, curing and environment, respectively.

The above equations 7 and 8 include parameters for defining the environmental loading, C_{sn}, and the resistance of the cover concrete, x, D_{ca} and n, and are based on diffusion theory, being a solution to Fick's second law of diffusion. The values

C_{sn}, D_{ca} and n can be determined by analysis of chloride profiles obtained from structures or field trials, or from laboratory tests. The constants can be derived from observations and analysis of data. To develop this equation it has been necessary to make the following simplifying assumptions;

i) Chloride ingress is principally by diffusion - while, in reality, chloride ingress involves a complex interaction of mechanisms (absorption, diffusion, binding), in many conditions the shape of the observed chloride profile can be fitted using diffusion theory and in the long term, as chlorides penetrate deeper, diffusion becomes the dominant mechanism.

ii) Spatial and temporal changes in the resistance of the concrete can be accommodated using the age factor, n.

iii) The surface chloride level remains constant after initial exposure - in the most severe exposure conditions this has been observed[7] while in less severe conditions, where chlorides may build up more slowly, it represents a safe assumption.

A spreadsheet model, AGEDDCA, has been developed[8] based on equation 7 and other relationships derived from an extensive review of published data [9]. While the model has limitations it provides a basis for predictions which are consistent with observations and which can be used for new or existing structures to make an estimate of the time to corrosion activation. It also provides a tool for scoping studies to determine which factors are most critical and which combinations of concrete and cover are required to achieve an acceptably low risk of corrosion.

The importance of taking account of variability can be demonstrated by a simple example. Consider an element in one of the most severe, splash zone exposure conditions. UK codes require the use of a low w/c ratio concrete (<0.45) with a high cement content (>400 kg/m^3). The time to achieve a chloride threshold level of 0.4% (cement weight) has been calculated using both typical values of C_{sn} and D_{ca} and upper 80 percentile values. The results are given in Table 1, together with the assumptions used in the calculations.

Table 1 *Calculation of time to corrosion activation for Portland cement concrete with a w/c of 0.45 in a marine environment*

Cover depth (mm)	Typical (Case 1)	Design (Case 2)
	Csn = 0.36 Dca = 9.4×10^{-13} m^2/s n = -0.264	Csn = 0.58 Dca = 1.47×10^{-12} m^2/s n = -0.264
25	4 years	1 year
50	26 years	9 years
75	77 years	26 years

The differences between the typical case and the (more extreme) design case are substantial and demonstrate the importance of selecting appropriate values for the input parameters.

To demonstrate the effect of variability of the input parameters more clearly, a numerical procedure has been adopted [1]. This involved generating a distribution of values for both C_{sn} and D_{ca} based on defined mean values and SD's and then calculating values of C_x for all possible combinations of C_{sn} and D_{ca}. For each individual distribution 35 values were generated hence, in combination, 1225 values of C_x were obtained. This calculation was repeated at different time intervals and typical frequency distributions are shown in Figure 3. As time proceeds, the distribution curve becomes flatter as high levels of chloride reach an increasing proportion of the reinfocement.

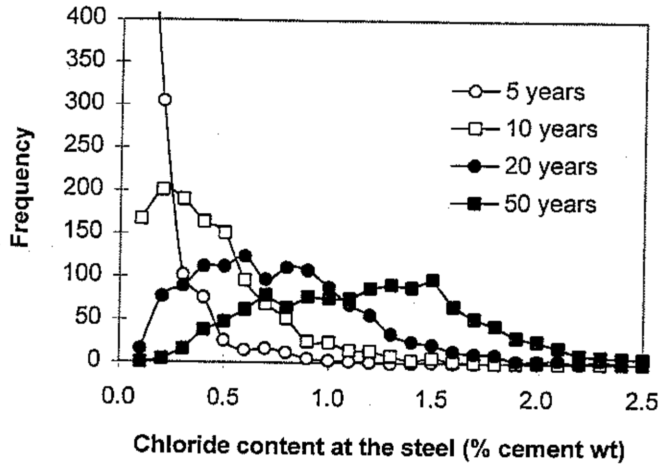

Figure 3 *Calculated distribution of chloride levels and its variation with time*

The data can also be presented to show the probability of different threshold values being exceeded and how this probability changes with time (Figure 4). It can be seen that, regardless of whether the threshold level is assumed to be 0.4% or 1.0%, there is still a significant risk of these values being exceeded after a relatively short period of exposure in relation to normally expected design lives (after 20 years, the probabilities are about 75% and 20% respectively).

The threshold level is also subject to variability and uncertainty. For example, a comprehensive study of bridges in the UK [10] involving several hundreds of observations showed evidence of corrosion even at chloride levels in the range 0.2% to 0.35% wt of cement. However, at some locations with much higher chloride levels (exceeding 1.5%) no corrosion was observed. The data are illustrated in Figure 5 and can be represented by a normal distribution with a mean of 1.1% chloride and a Standard Deviation of 0.6%. Laboratory research in

Germany [11] expressed the results in a similar manner but in this case the mean was 0.47% and the SD was 0.2%. It is interesting to note that, while the curves differ significantly at the higher levels of chloride, the value of chloride which represents a 5% risk is low in each case being 0.21% and 0.33% for the lab and site conditions respectively. This indicates the importance of adopting characteristically low values for design, as there is a very real risk of corrosion even at relatively low chloride levels. At the commonly assumed threshold of 0.4%, the respective risk levels for lab and site were 36% and 12% and these increased to 100% and 42% respectively at a chloride level of 1%.

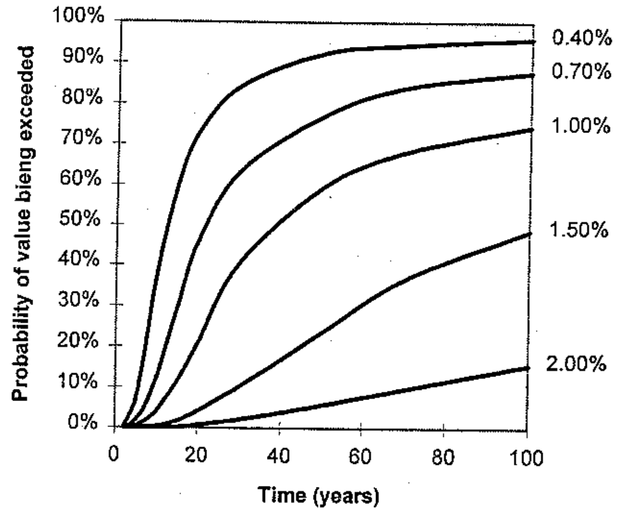

Figure 4 *Calculated probabilities of exceeding different chloride threshold levels*

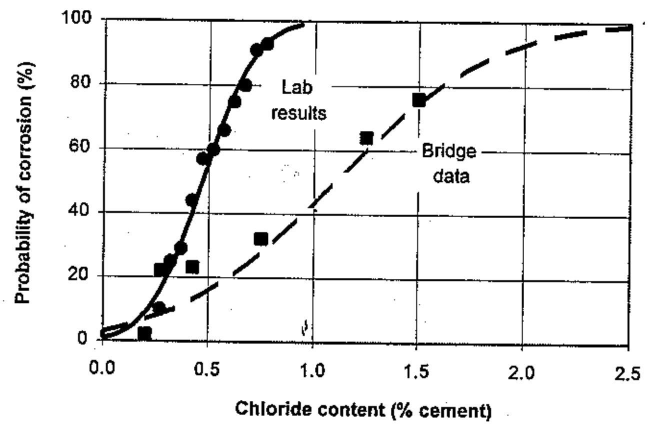

Figure 5 *The risk of corrosion at different levels of chloride obtained in the laboratory [11] and from bridge surveys [10]*

Advantages of probabilistic analysis

Adopting the probabilistic approach explains why single observations of apparently similar structures exposed in apparently similar environments can vary widely. They are all part of the same population but at different points within the overall distribution. This also explains why the relative performance of different materials or systems applied to concrete structures may differ between structures and between researchers.

Probabilistic analysis also enables a rational definition of design life based on the time for the risk of a defined serviceability limit state to be reached (Figure 6).

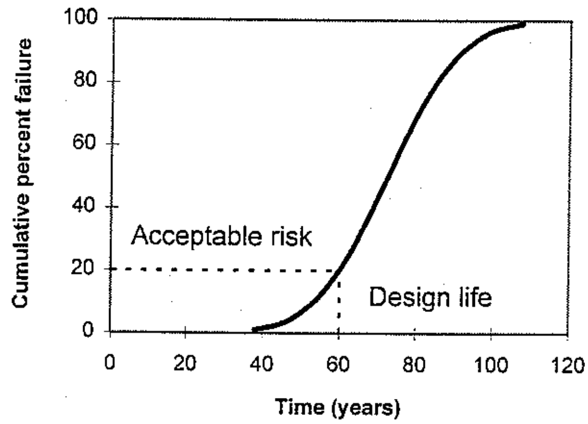

Figure 6 *A probability curve for a structure with a 60 year design life and an acceptable risk of exceeding the defined serviceability limit state of 20%.*

The influence of changes in specification and design can be presented in terms of the change in risk, enabling the Client and the Designer to understand the implications and to make decisions accordingly. For example, the performance of different systems for enhancing durability can be assessed either by comparing the relative levels of risk after a defined period, e.g. the design life, or by comparing the relative times for the level of risk to exceed a defined limit, e.g. 5% of the reinforcement becoming active or 2% of the surface requiring repair.

Use of Probabilistic Analysis in Life Cycle Costing

There is a growing recognition of the need for Life Cycle (or Whole Life) Costing (LCC) in order to demonstrate to clients the value of designing for greater durability and hence the benefit to society in the long term by reducing the utilisation of raw materials and the energy required in their conversion to building materials and components.

Life cycle costing involves estimating the total costs associated with construction, operation, maintenance, repair and demolition. To take account of the fact that different operations will take place at different times, incremental costs are converted to current costs using a discounted cash flow system. This takes account of interest rates and inflation through the discounted cost.

At its simplest level, therefore, LCC is relatively straightforward. If the cost in year t is equal to C_t, and the discount rate is r, then the life cycle cost for a structure with a design life of N years, expressed as the cost at current value, is as follows:

$$\text{Present Cost} = \sum_{t=0}^{t=N} \frac{C_t}{\left(1 + \frac{r}{100}\right)^t} \qquad (9)$$

To carry out a life cycle cost analysis it is necessary to make predictions about the long term performance of a building or structure. In particular values must be ascribed to the following;

1. The capital cost of construction
2. The cost of routine maintenance
3. The rate of deterioration
4. The level of deterioration at which intervention is required
5. The cost of repairs
6. The cost of lost production during the repair process
7. The rate of deterioration of the repairs
8. Any other costs resulting from the need to maintain and repair the structure

In addition, it is necessary to predict both the interest rate and the inflation rate in order to calculate the discounted costs based on the future value of money. The discounted cost rate, r, is calculated as follows;

$$\text{Discounted cost rate, r} = \frac{(1 + \text{interest rate})}{(1 + \text{inflation rate})} - 1 \qquad (10)$$

The LCC calculations are, therefore, dependent on the reliability of numerous assumptions, each of which is subject to a degree of uncertainty.

Current LCC models tend to make simplifying assumptions based on observations of performance and engineering approximations of deterioration rates. The extent of damage likely to have occurred after a specific period is, thus, estimated, e.g. after 20 years of exposure repairs will be required to (say) 5% of the surface. It is also commonly assumed that repairs will be required at fixed intervals, e.g. every 20 years. In the absence of extensive performance data from structures or validated predictive models such assumptions cannot be avoided. The probabilistic approach provides a means for quantifying the rate and extent of deterioration and hence a basis for estimating the cost of intervention. The output from the probabilistic model, i.e. the probability curve (Figure 6), becomes the input to the LCC model.

Input Data

As described above, the performance of a particular structure or element (as affected by in-situ concrete quality and cover) in a particular environment can be defined by a distribution function (or histogram) and a probability function (or cumulative distribution function). The example in Figure 7 shows the (normal) distribution functions for a structure with a design life of 60 years at which time no more than 5% of the structure should have exceeded its serviceability limit state. It is assumed that the Standard Deviation on the service life is 30 years and this results in the requirements for a mean service life (when the risk of failure is 50%) of 109 years.

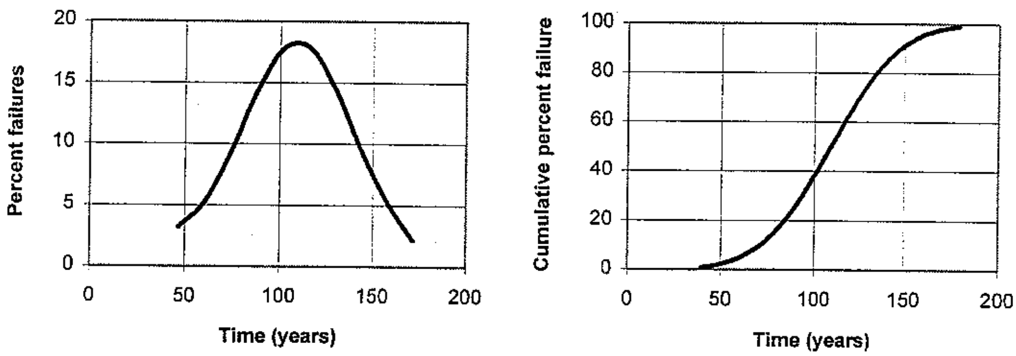

Figure 7 *Distribution functions for service life*

Presenting the time to achieve the serviceability limit state as a cumulative distribution function enables the time increments to be determined at which intervention is required. For a normal distribution, the rate of deterioration increases up to the time at which 50% has exceeded the defined limit state. As most structures will be designed such that the level of deterioration will be much less than 50%, then this will apply to all properly designed structures. The common assumption of a uniform deterioration rate in LCC calculations must, therefore, be reconsidered. Using the example in Figure 7 and assuming intervention is required at time intervals representing 2% increments of failure, calculated periods to intervention are as follows;

Percent failure	**1%**	**2%**	**3%**	**4%**
Years to intervention	**40**	**48**	**53**	**56**

While the first repairs are not required for 40 years, subsequent repairs, at the same magnitude, would be required at reducing intervals. The significance of this in relation to planned maintenance and repair is clear.

A spreadsheet model for life cycle costing

A spreadsheet model, LCCMODEL, has been developed which uses the probabilistic approach to predict the rate of expenditure on repair by using as input data a probability function to define the design life and its variability. Other input data are similar to those required for existing LCC models as follows:

Financial data - Average values of Interest Rate and Inflation Rate over the life of the structure are assumed for the calculation of discount rate using equation 10.

Capital costs - The capital cost is calculated from the total volume of concrete and reinforcement and its cost of placement (including formwork). The cost of the reinforced concrete is estimate by volume but the formwork (and later in the calculation the repair costs) are estimated by surface area. It is necessary, therefore, to calculate the surface area to volume ratio. This will depend on the geometry of a particular element. For example, a 500 mm column will have a SA/volume ratio of 8, while a 500 mm thick slab exposed on only one face will have a SA/volume ratio of 2.

Time dependent costs - The time dependent costs are broken down into the following elements;

Operating costs - these are the normal running costs of the building or structure and include planned maintenance.

Repair costs - these are the costs of completed repairs. The time at which repairs are required are predicted using the probabilistic analysis.

Lost production - these are the costs incurred as a result of the need to undertake repairs, for example, the cost of decanting staff and rental of alternative premises or the cost of a road closure.

Other costs - these are costs which are not covered by any of the above.

Costs relating to repairs are determined in time increments which are derived from the probabilistic analysis of deterioration. The design life is defined as the time at which the risk of achieving a specified serviceability limit state exceeds a defined level and the level of (risk of) deterioration at which intervention is required must also be defined. For example, action may be required when it is estimated that 5% of the reinforcing steel has become active or when cracking is observed over 2% of the surface.

Thus the required performance of the structure in relation to a particular serviceability state is defined together with the periods after which interventions are needed. These periods are currently defined by equal increments of deterioration (e.g. after 1%, 2%, 3%, etc.) From these input parameters the times are calculated at which defined levels of intervention are needed and these times are then fed into the cost model. If the first intervention is required when the risk level is 1% and the estimated time to reach this level is, say, 40 years then the model assumes that 1% of the surface will be in need of repair after this time. An example from the spreadsheet is shown in Table 2.

Table 2 *Calculated times to repair and the associated costs based on intervention at 1% increments of (risk of) damage*

REPAIR COSTS

New repairs

Constant costs: yes
Cost per square m: £600

% def.	Age	Repaired area	Current costs per unit area (Var)	(fixed)	cost	Disc. cost
1.0	40	558	400	600	334950	51237
2.0	48	558	400	600	334615	34871
3.0	53	558	500	600	334615	27384
4.0	56	558	500	600	334615	23088
5.0	60	558	600	600	0	0
6.0	62	558	600	600	0	0
7.0	64	558	600	600	0	0
8.0	66	558	600	600	0	0
9.0	69	558	600	600	0	0
10.0	71	558	600	600	0	0
						136581

In addition to predicting the rate of deterioration of the structure, a similar approach is use to predict the performance of the repairs which may themselves deteriorate within the design life.

The life cycle cost

Having calculated the capital cost and the incremental costs of operation, repair, etc. the costs are accumulated to produce a life cycle cost curve and the total cost at the end of the design life (Figure 8).

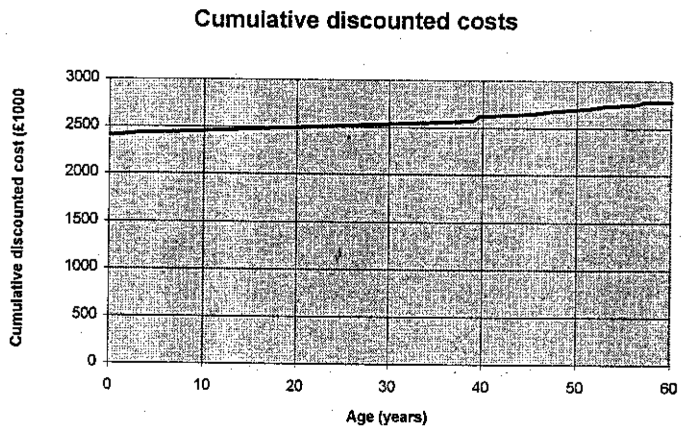

Figure 8 *Cumulative discounted cost curve*

Conclusions

The current approach to durability design is limited in many respects and premature deterioration of reinforced concrete resulting from reinforcement corrosion continues to be a major problem. Codes use a deemed-to-satisfy approach but in many cases fail to define either service life or serviceability limit states relating to durability. It is, therefore, difficult within the current design framework to quantify predicted performance and hence to evaluate the cost-effectiveness of proposed improvements.

A new approach to durability design is proposed which uses probabilistic analysis. This enables the changing state of a structure to be predicted and the probability of defined durability limit states being exceeded to be quantified. The approach is applicable to new structures, to enable whole life cost optimisation, and to existing structures, for optimisation of repair and future maintenance.

By defining the risk of deterioration and the associated costs the Client can make decisions about different design options on a more rational basis. The probabilistic approach may also provide a basis for risk sharing and for optimising insurance premiums.

Acknowledgements

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