Probability Based Chloride Diffusion Model to Predict the Condition States of RC Bridge Elements

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ABTRACT

Major cause of deterioration of reinforced concrete bridge components is corrosion of reinforcement due to chloride ingress. This paper presents a probability-based model to assess the deterioration of reinforced concrete elements due to chloride ingress. The aim of the model is to estimate the lifetime of bridge components incorporating the time to corrosion initiation, crack initiation and crack propagation models. The model is based on probability distribution of chlorides, concrete types and cover to reinforcement of the bridge elements in a particular region under consideration. The analysis is based on time varying diffusion parameters, which gives better results than the commonly used simplified error function formula. The crack initiation and propagations are then related to the condition states defined by VicRoads bridge inspection manual. This manual is used to assess the condition states of the existing bridge stock. The condition states of RC elements are defined based on cracking and spalling of concrete, and percentage loss of area of the reinforcement.

The application of this methodology is illustrated using chloride profiles for a bridge deck component along coastal regions of Victoria. The results are compared with the actual inspection data of the bridge components made available from VicRoads for this region. The comparison demonstrates that the future condition states of the bridge components can be estimated using the chloride diffusion model presented in this paper. The ability to predict the future condition states of bridge components based on rational model is an essential tool in a bridge management system.

1 INTRODUCTION

The corrosion of steel in concrete is an electrochemical process. The risk of corrosion is minimal in a well-designed reinforced structure where the concrete cover provides the physical barrier. The high alkalinity of concrete pore water provides the chemical barrier by forming a passive layer on the steel surface [1]. The corrosion of the steel may begin when the chemical barrier ceases to be effective and the passivating film becomes unstable. This 'depassivation' can occur by the effect of chloride ions that can induce corrosion. Chloride ions act as catalysts to corrosion when there is sufficient concentration (above 0.4% by weight of cement) at the reinforcing bar surface to breakdown the passive layer. Damage caused by steel corrosion can be summarized as (a) Rust staining (b) Delamination (c) Cracking (d) Spalling (e) Loss of serviceability (f) Ultimate failure of the structure.

Damage due to corrosion of steel in concrete can be modelled as initiation and deterioration stages. The diffusion of chloride ions to the level of the reinforcing bars leads to the initiation stage, and time to complete this stage is known as initiation time t_o. The next stage is deterioration when delamination, cracking and spalling of the concrete occur. Time for the deterioration stage is known as propagation time. Figure 1 shows the deterioration model of reinforced concrete elements due to corrosion of steel.

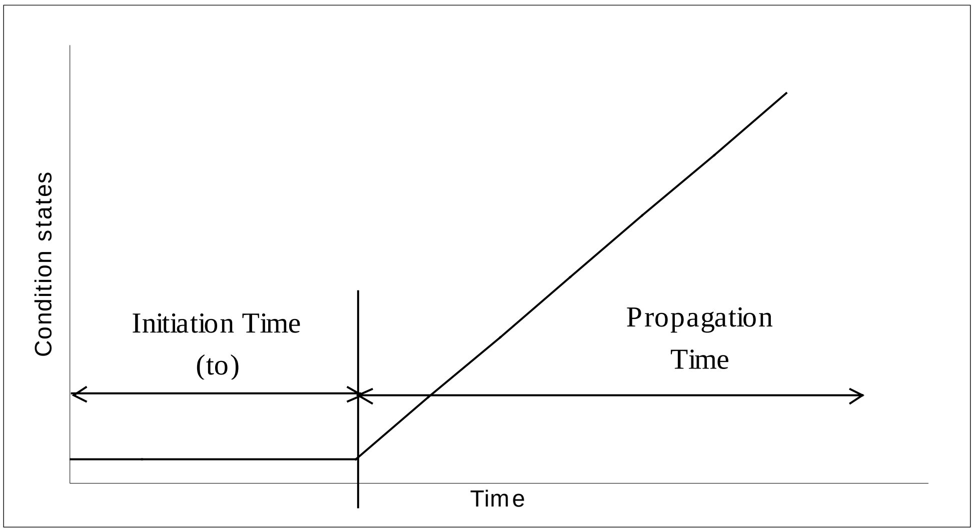


Figure 1: Deterioration model of reinforced concrete elements due to corrosion

The deterioration due to the chloride-induced corrosion can be estimated using corroison initiation, and crack initiation and propagation models. Life-365 model [2] is used in this paper to calculate the percentage chloride concentration at various depths. This model uses time-dependent diffusion coefficient (D) and surface chloride concentration (C_0) that gives more accurate results than models based on constant D and C_0. Bamforth's model [3] and Rodriguez's model [4] are used for crack initiation and propagation.

2 MODELLING OF DETERIORATION OF CONCRETE DUE TO STEEL CORROSION

2.1 Corrosion initiation (Diffusion models)

There are a number of different models available to predict the chloride diffusion in concrete, based on varying levels of simplifying assumptions. The commonly used ones are presented below:

2.1.1 Fick's law of diffusion

Fick's second law of diffusion deals with one-dimensional diffusion and given by,

$$\frac{\partial C}{\partial t} = D\frac{\partial^2 C}{\partial x^2} \tag{1}$$

For one-dimensional flow into a semi-infinite medium a closed-form solution can be derived for equation (1) as follows,

$$C(x,t) = C_0\left(1 - erf\,\frac{x}{2\sqrt{(D\,t)}}\right) \tag{2}$$

where, $C(x,t)$ = concentration at depth x (m), at time t (% by mass); t = time of exposure (seconds); C_0 = surface chloride concentration (% by mass); D = constant diffusion coefficient (m^2/s); erf = error function.

The equation (2) can be used only if the diffusion coefficient, D, and surface chloride concentration, C_0 remain constant during the time of diffusion. It has been shown by previous researchers [2, 5, 6] that these conditions do not apply in practical conditions. The equation (1) can be modified to take account of the variation of D with time as shown by Crank [7]:

$$\frac{\partial C}{\partial t} = D(t)\frac{\partial^2 C}{\partial x^2} \qquad (3)$$

Finite difference method can be used to solve the equation (3).

2.1.2 Life-365 model

Life-365 [2], a computer program for predicting the service life and life-cycle costs of reinforced concrete exposed to chlorides was developed by a consortium established under Strategic Development Council of the American Concrete Institute. This software presents an initial life cycle cost model based on existing service life model developed at the University of Toronto [8]. The version 1.0 represents the first phase of a long-term goal to develop a comprehensive service life and life cycle model for reinforced concrete. The current version has many limitations in that a number of assumptions or simplifications have been made to deal with some of the more complex phenomena or areas where there is insufficient knowledge to permit a more rigorous analysis.

Life-365 software uses finite difference method to solve the equation (3). The following relationship is used to account for time-dependent changes in D,

$$D(t) = D_{ref}\left(\frac{t_{ref}}{t}\right)^m \qquad (4)$$

where, $D(t)$ = diffusion coefficient at time t; D_{ref} = diffusion coefficient at some reference time t_{ref} (=28 days); m = constant (depending on concrete mix proportions).

2.2 Crack initiation and propagation models

Once the corrosion has been initiated by chloride contamination (above 0.4% by cement mass), loss of steel sections occur at a rate dependent on various parameters. The time to cracking can be determined by the rate of corrosion and the ability of the concrete surrounding the reinforcement to accommodate the corrosion products. The volume corrosion products will be as high as 2 to 4 times that of the un-corroded steel.

2.2.1 Rodriguez's model

Rodriguez et al [4] proposed the following equation to calculate the corrosion penetration depth corresponding to crack initiation in microns,

$$y_0 = a + b_1\frac{c}{\phi} - b_2 f_{st} \qquad (5)$$

where, y_0 = corrosion penetration depth required for crack initiation; $\frac{c}{\phi}$ = cover to reinforcement / diameter ratio; f_{st} = splitting tensile strength of the concrete in MPa; a, b_1, b_2 = constants as given in Table 1.

The following equation estimates the crack width in mm,

$$w = 0.05 + \beta(y - y_0) \qquad (for\ w \le 1.0\ mm) \qquad (6)$$

where, w = estimated crack width in mm; y = penetration depth or the amount of steel lost in microns; β = constants as given in Table 2.

Table 1: Parameters a, b_1, b_2

Parameter	Mean	Standard deviation
a	74.5	5.64
b_1	7.3	0.06
b_2	17.4	3.15

Table 2: Parameter β

	Mean	Standard deviation
Top-cast steel	0.0086	8.5E-04
Bottom-cast steel	0.0104	1.3E-03

2.2.2 Bamforth's model

Bamforth [3] derived a relationship between corrosion rate and chloride content based on tests performed for six years of exposure of samples as described below:

$$CR = -6.0417 + 8.786\, C_x \qquad (7)$$

Other researchers [9, 10] also observed a relationship between corrosion rate and chloride levels at reinforcement level. A detailed analysis of the results by Bamforth [3] indicated that the relationship is not linear, but exponential and the following equations are recommended for various exposure conditions. The corrosion rate for moderate exposure conditions where the concrete is wet and rarely dries is as follows:

$$CR = 0.84\, e^{0.64 C_x} \qquad (8)$$

For severe exposure conditions where the concrete subjected to airborne sea water and cyclic wet / dry:

$$CR = 0.54\, e^{1.56 C_x} \qquad (9)$$

For very severe exposure conditions where the concrete is in tidal zone:

$$CR = 0.46\, e^{1.84 C_x} \qquad (10)$$

where, CR = corrosion rate (microns/ year); C_x = chloride at reinforcement level (% by wt of cement)

2.2.3 Liu and Weyers's model

Liu and Weyers [11, 12] experimental study showed that the critical amount of corrosion products required to induce cracking of the cover is mainly dependent on the tensile strength of concrete, cover depth, elastic modulus of the concrete, and properties (void structure) of steel /concrete interface. The critical amount of corrosion products to induce cracking of the cover concrete can be estimated from the following equation;

$$W_{crit} = \rho_{rust}\left(\pi\left[\frac{Cf'_t}{E_{ef}}\left(\frac{a^2+b^2}{b^2-a^2}+\nu_c\right)+d_o\right]D+\frac{W_{st}}{\rho_{st}}\right) \qquad (11)$$

where, $\rho_{rust,}$ ρ_{st} = density of rust product and original steel, respectively; d_o = thickness of pore space around bar; C = cover depth; D = bar diameter; a = distance from bar center to outer pore space ($D/2 + d_o$); b = distance from center of bar to concrete surface (a + C); W_{st} = weight of original steel consumed by corrosion; ν_c = Poisson's ratio for concrete; f'_t = tensile strength of concrete; E_{ef} = effective modulus of elasticity of concrete, $\left(\dfrac{E_c}{1+\varphi_{cr}}\right)$, where E_c = elastic modulus of concrete; φ_{cr} = creep coefficient of concrete.

The time to cracking can be estimated as follows,

$$t_{cr} = \frac{W_{crit}^2}{2k_p} \tag{12}$$

$$k_p = 0.098\left(\frac{1}{\alpha}\right)\pi\, D\, i_{cor} \tag{13}$$

where, t_{cr} = time to cracking (for a constant corrosion rate); k_p = rate of rust production; α = ratio of mol.wt.of steel to mol.wt. of corrosion product; i_{cor} = annual mean corrosion current density.

Liu and Weyers's model gives only the time to cracking and the parameters required to use this model are not readily available. Therefore, Rodriguez's model and Bamforth's model described in sections 2.2.1 and 2.2.2 are used in this paper.

3 PROBABILITY BASED MODELLING OF STEEL CORROSION

3.1 Probabilistic model for chloride diffusion

The parameters of interest in engineering analysis have some degree of uncertainty and thus may be considered to be random variables. In general, repeated measurements of physical phenomena generate multiple observations giving the basic data to formulate appropriate distribution function for the physical phenomena. The probabilistic methods to estimate the time for deteriorations can be divided into two main categories [13]: implicit and explicit.

Implicit probabilistic methods directly integrate equations for probability density functions into the equations modelling chloride transport. These methods usually result in a set of equations that directly predict the probability of corrosion at a given time. As an example, the Life-365 [2] primary modelling equations directly include the parameters in service life modelling. The reliability methods such as first-order reliability method (FORM) and second-order reliability method (SORM) can be classified as explicit probabilistic methods. The Monte Carlo simulation is also an explicit probabilistic method that is used in this paper to predict future condition states of bridge elements. The Monte Carlo simulation technique [14] has six essential elements: (a) define the problem in terms of all the random variables (b) quantify all probabilistic characteristic of the random variables (c) generate random values of these variables (d) evaluate the problem deterministically for each realisation of random variables (e) extract probabilistic information from each evaluation of the problem (f) determine the accuracy and efficiency of the simulation.

3.2 Input parameters

3.2.1 Surface chloride concentration C_s

Surface chloride concentrations vary depending of the location of the structure (macroclimate) as well as within different parts of the same structure (microclimate). A dramatic variation can be expected even within a structure where the chloride contamination is due to the natural causes. The variation from city to city in USA and Canada is given in Life-365 [2]. The statistical parameters for the surface chloride concentration for this paper were derived using the chloride profiles totalling 73 numbers collected by VicRoads (State authority responsible for managing bridge assets in Victoria, Australia) along coastal areas of Victoria, Australia. The surface chloride concentration for each profile was calculated by fitting the most probable profile curve to field data. The finite difference method was used to solve the equation (3) where diffusion coefficient is varying as per equation (4). The Figure 2 shows the sample field data and the best-fit chloride profile curve. Once the surface chloride concentrations for all the profile have been calculated, a probability distribution function was estimated using a commercially available statistical computer software SPSS [15].

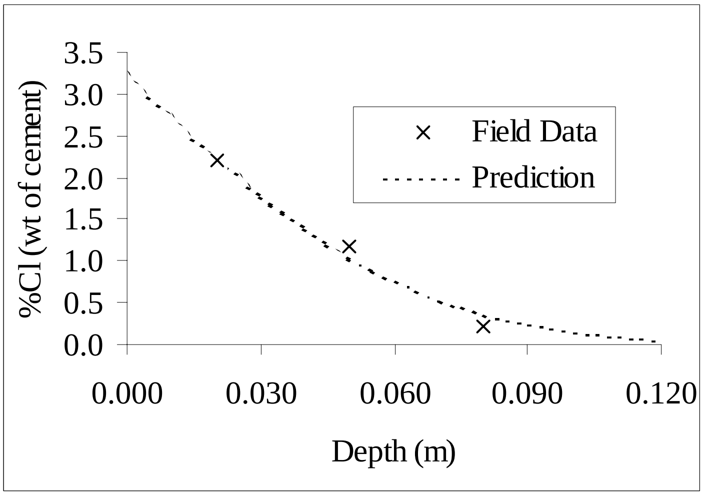


Figure 2: Sample field data and fitted chloride profile curve

3.2.2 Diffusion coefficient D_{ref}

The diffusion coefficient depends on various parameters. Life-365 [2] assumes that the diffusion coefficient depends on water/cement ratio (w/cm) and uses the following equation to estimate the value for Portland cement concrete,

$$D_{PC} = 10^{(-12.06+2.4w/cm)} \tag{14}$$

The addition of silica fume to concrete is known to produce reductions in the permeability and diffusivity of concrete. The following relationship is recommended in Life-365 to account for the reduction based on the level of silica fume (%SF) in the concrete,

$$D_{SF} = D_{PC}\ e^{-0.165\,SF} \tag{15}$$

Bamforth [5] recommended mean apparent diffusion coefficient values ranging from $0.63\,x\,10^{-12}$ to $15.5\,x\,10^{-12}$ for concrete based on an eight-year study of UK coastal exposure trials. The diffusion coefficient data was calculated for this paper using the chloride profiles obtained from various structures by VicRoads. A typical fit for a chloride profile data to obtain the diffusion coefficient is shown in Figure 2, which is the same fit used for the surface chloride data.

3.2.3 Slope of diffusion log plot m

There is no clearly defined test procedure to evaluate slope of the diffusion log plot. The values recommended in Life-365 [2] and Bamforth [5] vary from 0.2 to 0.7. Shayan [16] reports that the laboratory concrete samples give a value from 0.4 to 0.8. The slope of diffusion log plot m can be estimated using the curve fitting technique if the chloride profile data is available for the same structure at different point in time. The chloride concentration obtained for various bridges by VicRoads was done only at one point in time. A base value of 0.2 is recommended for Portland cement concrete in Life-365. Therefore, a constant value of 0.2 is used in this paper in the absence of accurate estimation.

3.2.4 Chloride threshold level C_t

There is no standard test procedure to determine the chloride threshold level that causes the corrosion initiation in concrete. Bamforth [5] research shows that a significant difference between the performance of the different concretes, indicating unique threshold value does not exit. The research further shows that the chloride level less than 0.5% (by wt of cement) gives a negligible corrosion risk and more than 1.5% (wt of cement) gives a high risk of corrosion. Life-365 uses a default value of 0.05% (by wt of concrete) as chloride threshold level. Glass and Buenfeld [17] research shows that the threshold value can vary between 0.03% to 0.07% (by wt of concrete). A chloride threshold value ranging from 0.2 to 0.4 (by wt of cement) is taken as reasonable and used in the analysis.

3.2.5 Clear cover to reinforcement c

The depth of clear cover can vary depending on quality of design and specification details and quality of construction. McGee [18] produced extensive collection of cover data for Tasmanian bridges. Table 3 shows the results of McGee's analysis of the data.

Table 3: Cover model for analysis

Element type	Mean Cover	σ
Cast on site	Specified + 6 mm	11.5 mm
Precast	Specified + 3 mm	9.7 mm
Culvert	Specified	3.6 mm

It is estimated that the cover distribution in Victoria would also follow McGee's results. Therefore, the clear cover distribution is taken as per Table 3 in the analysis.

3.3 Condition states of bridge elements

3.3.1 background

VicRoads has acquired a large amount of bridge inspection data since introducing a regular bridge inspection program in 1996. VicRoads bridge inspection policy [19] requires that the bridge condition inspections be carried out on three levels to assess the condition of each structure and its principal components. Level-1 inspections are routine maintenance inspections carried out in conjunction with routine pavement maintenance on a 6 monthly frequency to check general serviceability of the structure for road users. Level-2 inspections are managed on a state-wide basis to assess the condition state of each structure and its principal components. The frequency of inspection varies between 2 to 5 years depending on bridge rating. The bridge element condition state is described on a scale of 1 to 4, where 1 stands for "excellent condition" and 4 stands for "serious deterioration". The inspector records the condition states of the bridge element and the percentage of that element in a particular condition state. Level-3 inspections are detailed engineering inspections conducted on a needs basis to assess the structural condition and capacity of structures that have been identified as potential candidates for rehabilitation, strengthening, widening or replacement. Level 2 inspections are used in this paper to compare the results from diffusion model.

3.3.2 Condition guidelines for precast slabs (Element 8P)

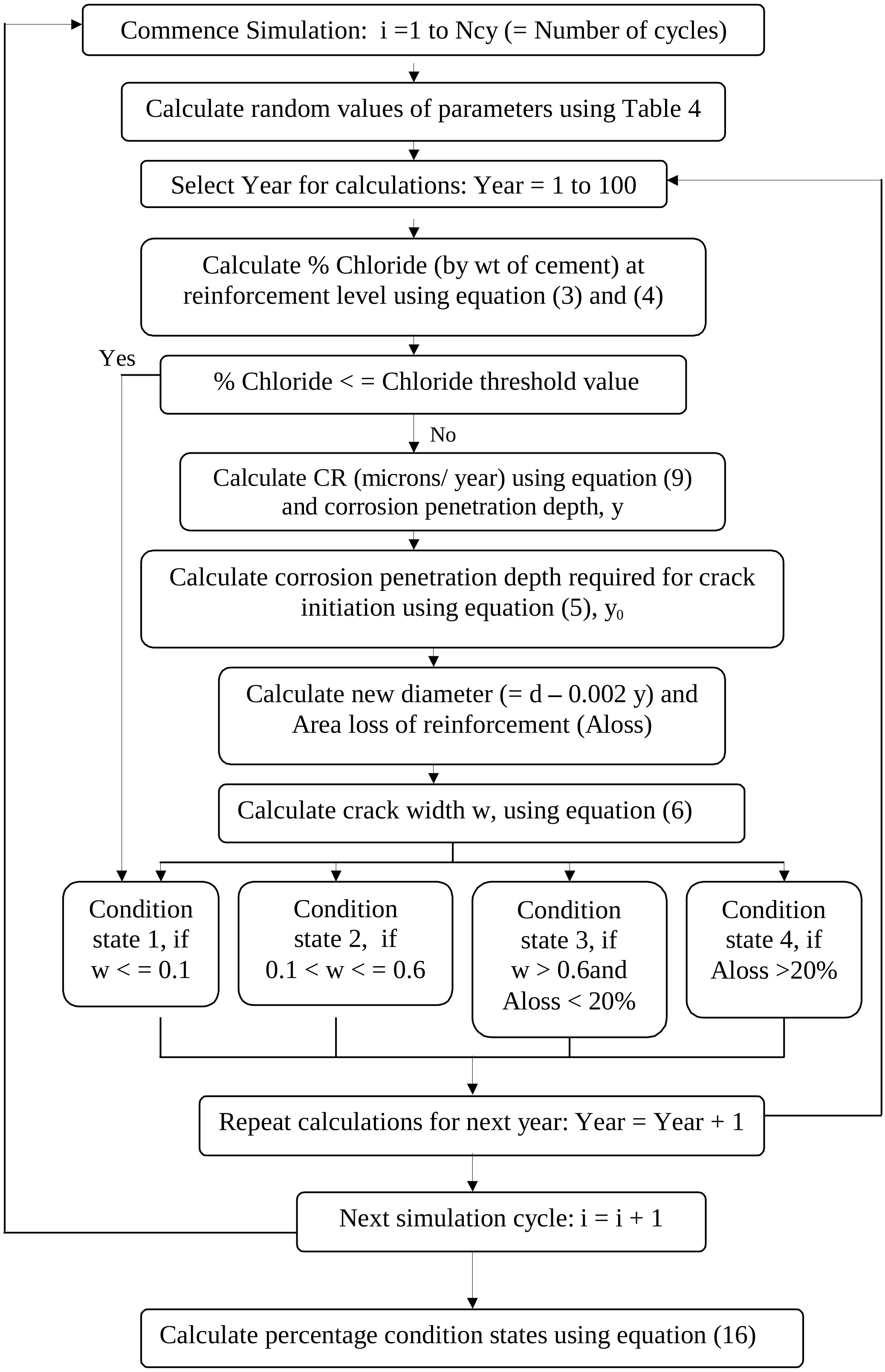
The percentage condition state 1 (pc1) as defined in VicRoads bridge inspection manual [19] allows only minor faint cracking or minor edge chipping whereas the pc2 allows minor reinforcement corrosion. The amount of corrosion allowed in pc2 is not specified, but the parts of element in pc1 or 2 do not require any repair works. The percentage condition state 3 allows medium cracking and spalling with up to 20% loss of reinforcement section. The percentage condition state 4 allows heavy spalling and advanced corrosion and can be assumed as more than 20% loss of reinforcement section.

3.4 Application of the methodology

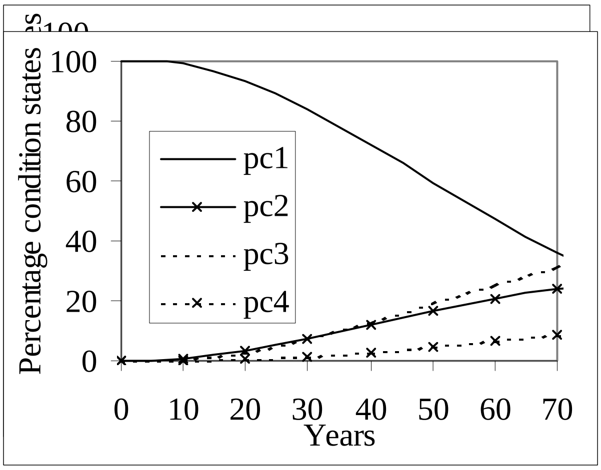
Table 4: Parameters used for simulation *(* numbers in natural log value)*

Parameters	μ **(Mean)**	σ **(SD)**	**Distribution**
Surface chloride concentration (by wt of cement)	$\alpha = 1.93$	$\beta = 0.82$	Gamma (mean = 1.97)
Diffusion coefficient (m^2/s) ($D_{ref}\, x10^{12}$)	5.31E-12 *-25.96**	*0.74**	LogNormal
Slope of diffusion log plot (m)	0.2	-	Constant
Chloride threshold (C_t)	0.3	-	Uniform (0.2 – 0.4)
Cover c (truncated at 10 mm)	63	9.7	Normal
Concrete compressive strength (f_{cu} MPa)	37.5	3.33	Normal
a	74.5	5.64	Normal
b_1	7.3	0.06	Normal
b_2	17.4	3.15	Normal
β - top cast	0.0086	8.5E-04	Normal
β - bottom cast	0.0104	1.3E-03	Normal

Figure 3. Flow chart for application of methodology



The flow chart in Figure 3 shows the application of the methodology. The random parameter values calculated using Table 4 for a simulation cycle in Figure 3 assumed to be belonging to one point in a bridge element. The calculations are repeated for each year and the condition state of that point at each year is stored in a file. Once the calculations for required number of years are completed, next simulation cycle (assumed to be another point in a bridge element)



is commenced and calculations repeated for 50,000 simulation cycles. The condition state of the element at a given time can be calculated using the following equation,

$$\text{Percentage condition state } i = \frac{100\, n_i}{50{,}000} \tag{16}$$

where, $i =$ 1 to 4; $n_i =$ the number of points satisfied the conditions as given in Figure 3.

4 RESULTS AND COMPARION WITH BRDGE INSPECTION DATA

Figure 4: Predicted condition states from chloride profile data

Figure 5: Predicted condition states from bridge inspection data (Element 8P-Uslabs)

Table 5: Comparison of percentage condition states

Year	Chloride profile data				Bridge inspection data (8P-Uslab, with no overlay)			
	pc1	pc2	pc3	Pc4	pc1	pc2	pc3	pc4
0	100	0	0	0	100	0	0	0
10	99	0.5	0.5	0	89	10	1	0
20	94	3	3	0	78	17	4	1
30	84	7	7	2	69	21	7	3
40	72	12	13	3	62	22	10	6
50	60	16	19	5	55	23	12	10
60	47	21	25	7	48	23	14	15
70	36	24	31	9	43	22	15	20

Figure 4 shows the deterioration of concrete bridge element due to chloride ingress in exposure condition 4. Figure 5 shows the predicted deterioration curves using bridge inspection data for U-slabs (with no overlay) in exposure condition 4. Since the bridge inspection data includes all forms of deterioration, the deterioration rate in Figure 4 can be expected to be slower than that was predicted using bridge inspection data. Figure 5 shows deterioration for only one type of slab where as the Figure 4 shows a general deterioration curves giving the average values over a number of elements and structures. The results beyond 5 – 10% of percentage condition state 4 (area loss of reinforcement is greater than 20%) have only theoretical value since it is expected that repair work be carried out at this stage to the bridge elements to improve the condition states.

Table 5 shows the comparison of percentage condition states. A close examination of pc4 based on chloride profile data and bridge inspection data reveals that the result based on chloride profile data is less conservative. This can be attributed to the fact that the defects detected from the bridge inspection data includes all forms of deterioration, that is: construction defects, overloading of bridge elements, differential movements of elements, shrinkage, and corrosion due to carbonation and chlorides whereas the deterioration predicted using chloride profile data contains only the chloride induced corrosion. Therefore, the condition states predicted from the chloride profile data can be treated as a lower bound solution.

5 CONCLUSIONS

1. Future condition states of concrete bridge elements can be predicted by the use of chloride diffusion calculations based on probability distributions.

2. An example calculation for concrete bridge element 8P shows that this methodology can be used for prediction of future condition states of the concrete elements. These predictions can be considered as a lower bound solution.

3. Research work required developing a clearly defined method to estimate the slope of diffusion log plot m and chloride threshold level.

4. It is recommended for further research on crack propagation mechanism since there is only limited literature can be found.

5. Further research work also required on relationship between corrosion rate and chloride concentration at reinforcement level.

6. Results beyond 5 – 10% of condition state 4 (area loss of reinforcement is greater than 20%) have only theoretical value since it is expected that repair work be carried out at this stage to the bridge elements to improve the condition states.

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REFERENCES

1. Bentur, A., S. Diamond, and N.S. Berke, *Steel Corrosion in Concrete: Fundamentals and Civil Engineering Practice*, E & FN SPON, 1997.
2. Thomas, M.D.A. and E.C. Bentz, "Life-365 (Ver 1.0.0) : Computer Program for Predicting the Service Life and Life-Cycle Costs of Reinforced Concrete Exposed to Chlorides", Version: 1.0.0, Master Builders Technologies, 2000.
3. Bamforth, P.B., "Guidance on the Selection of Measures for Enhancing Reinforced Concrete Durability: Appedix 2 - A Spreadsheet Model for Predicting Chloride Induced Corrosion and the Influence of Various Protective Measures", Report No: 1304/98/10439, Taywood Engineering Ltd, 1998.
4. Rodriguez, J.L., L.M. Ortega, and J. Aragoncillo, "Corrosion rate and structural performance", *Measurements and interpretation of the on-site corrosion rate - International workshop Mesina*, Instituto Eduardo Torroja, Madrid, Spain, 1999,
5. Bamforth, P.B., "The derivation of input data for modelling chloride ingress from eight-year UK coastal exposure trials", *Magazine of Concrete Research*, 1999, V. 51, No. 2, 87-96.
6. Kassir, M.K. and M. Ghosn, "Chloride-induced corrosion of reinforced concrete bridge decks", *Cement and Concrete Research*, 2002, V. 32, No. 1, 139-143.
7. Crank, J., *The Mathematics of Diffusion*, Oxford University Press, First Edition, 1956.
8. Boddy, A., et al., "An Overview and Sensitivity Study of a Multi-Mechanistic Chloride Transport Model", *Cement and Concrete Research*, 1999, V. 29, No. 6, 827-837.
9. Andrade, C. and C. Alonso, "Corrosion rate monitoring in the laboratory and on site", *Construction and Building Materials*, 1996, V. 10, No. 5, 315 - 328.
10. Goni, S. and C. Andrade, "Synthetic concrete pore solution chemistry and rebar corrosion in the presence of chlorides", *Cement and Concrete Research*, 1990, V. 20, No. 4, 525 - 539.
11. Liu, Y. and R.W. Weyers, "Modelling the Time-to-Corrosion Cracking in Chloride Contaminated Reinforced Concrete Structures", *ACI Materials Journal*, 1998, V. 95, No. 6, 675 - 681.
12. Liu, Y., "Modeling the Time-to-Corrosion Cracking of the Cover Concrete in Chloride-Contaminated Reinforced Concrete Structures", PhD Thesis, Department of Civil Engineering, Virginia Polytechnic Institute and State University, 1996.
13. Bentz, E.C., "Probabilistic Modeling of Service Life for Structures Subjected to Chlorides", *ACI Materials Journal*, 2003, V. 100, No. 5, 391 - 397.
14. Haldar, A. and S. Mahadevan, *Probability, Reliability, and Statistical Methods in Engineering Design*, John Wiley & Sons, 2000.
15. SPSS, "SPSS for Windows", Version: 10.0.5, SPSS Inc.,Chicago, USA, 1989 - 1999.
16. Shayan, A. and A. Xu, "Management of Concrete Bridge Structures to Extend their Service Life", Report No: AP - TP11, AustRoads, 2002.
17. Glass, G.K. and N.R. Buenfeld, "Chloride Threshold Levels for Corrosion Induced Deterioration of Steel in Concrete", *RILEM International Workshop on Chloride Penetration into Concrete*, RILEM Paris,1997, 429 - 452.
18. McGee, R.W., "On the Service Life Modelling of Tasmanian Concrete Bridges", PhD thesis, Civil & Mechanical Engineering, University of Tasmania, 2001.
19. VicRoads, *VicRoads Bridge Inspection Manual*, 1995.