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THESIS

OPTIMIZATION OF BRIDGE MANAGEMENT AND MAINTENANCE PLAN

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A Thesis Submitted in Partial Fulfillment of
the Requirements for the Degree of
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Surattachon Chauytook 2009: Optimization of Bridge Management and Maintenance Plan. Master of Engineering (Civil Engineering), Major Field: Civil Engineering, Department of Civil Engineering. Thesis Advisor: Assistant Professor Piya Chotickai, Ph.D. 136 pages.

Life-cycle cost analysis has become an important consideration in a maintenance and rehabilitation plan for highway bridge structures during the past decades to achieve an efficient means for allocating limited capital and funds on the preventive and maintenance program. Consequently, certain safety and performance levels of bridge structures can be obtained. Based on the structural reliability concept, a stochastic model for evaluating the level of safety and service life of bridge superstructure and substructure, which has deteriorated due to chloride-induced corrosion, is presented in this research study. The inherent uncertainties in material properties and environmental effects were incorporated in the model to determine the reduction in capacity due to loss in cross-sectional areas of reinforcing steels. The load and resistance parameters were treated as random variables with statistical parameters obtained from available literatures. The Monte Carlo simulation was utilized during the calculation process to obtain the probability of survival over the lifetime of the structures. Additionally, by employing the Fault Tree Decision process, the optimum maintenance strategy for an example of the deteriorated reinforced concrete bridges in the southern part of Thailand was determined. The flexural and compression capacities were used in the calculation to determine the life-cycle cost for the slab and pier, respectively. Analysis procedure developed in this study may be use to obtain efficient maintenance strategy of bridge structures deteriorated due to chloride induced corrosion.

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The author wishes to express sincere appreciation to his advisor, Dr. Piya Chotickai, and his committee, Dr.Prasert Suwanwitaya and Dr. Suneerrat Kusalasai, for their useful and valuable guidance and suggestions.

The author also expresses thanks to his parents and friends who have continuously given the best support of all kind and inspired him with affection.

Surattachon Chauytook

April 2009

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D_C	=	apparent diffusion coefficient, mm2/year
λ_2	=	bias factor of asphalt surface
λ_1	=	bias factor of cast-in-place concrete
λ_3	=	bias factor of miscellaneous
λ_{LL}	=	bias factor of live load
$C_{(x,t)}$	=	chloride concentration at depth and time, kg/m3
f_c'	=	compressive strength of concrete, ksc
cover	=	concrete cover depth, cm
x	=	concrete cover depth, mm
λ	=	corrosion rate at a surface, mm/year
$i_{\text{corr}}(1)$	=	corrosion rate at the start of corrosion propagation and cover, cm
C_{INSP}	=	cost of all inspections, US$
C_{REP}	=	cost of all repairs, US$
C_{FAIL}	=	cost of failure, US$
C_{IN_i}	=	cost of inspection at i
C_{R_i}	=	cost of repair at i
dc	=	covering of compression steel, m
dt	=	covering of tension steel, m
F_x	=	cumulative density function of variable x_i
Φ	=	cumulative density functions of the standard normal distribution
$\underline{u}^*$	=	design point $(u_1^*, u_2^*, ..., u_n^*)$
u_i^*	=	design point in the transformed coordinate system
ϕ_t	=	diameter of compression steel, mm
ϕ_c	=	diameter of tension steel, mm
$D(t)$	=	diameter of the reinforcing steel at time, mm

LIST OF ABBREVIATIONS (Continued)

C_F	=	failure cost coefficient, US$
λ_{IM}	=	impact factor (dynamic factor)
r	=	inflation rate of money
D_i	=	initial bar diameter, mm
C_{INIT}	=	initial cost, US$
Q	=	load demand on the component
μ_i^N	=	mean of equivalent normal distribution parameters
$\min C_T$	=	minimum cost of all repairs, US$
$t_{\min}$	=	minimum time between inspections
$F_x^/$	=	normal cumulative density function
$f_x^/$	=	normal probability density function
i	=	number of inspection/repair
f_x	=	probability density function of variable x_i
φ	=	probability density functions of the standard normal distribution
P_f	=	probability of failure
$P_f(75)$	=	probability of failure of structure member
P_s	=	probability of survival
β	=	reliability index
R	=	resistance of load carrying capacity of the structural component
δ, ε	=	specified tolerance values
σ_Q	=	standard deviation of load
σ_R	=	standard deviation of resistance
erf	=	statistical error function
C_O	=	surface chloride concentration, kg/m3
P_f^0	=	target lifetime failure probability
t	=	time for diffusion, year

LIST OF ABBREVIATIONS (Continues)

t_{R_i}	=	time of inspection/repair
t_{R_i}	=	time of repair, year
t_P	=	time since corrosion initiation
T_i	=	time to diffusion, year
C_T	=	total cost, US$
n	=	total number of inspection/repairs
x_i	=	value of the design point
$\frac{W}{C}$	=	water cement ratio
F_y	=	yield strength of steel, ksc

OPTIMIZATION OF BRIDGE MANAGEMENT AND MAINTENANCE PLAN

INTRODUCTION

Life cycle costing of highway bridge structures is required by highway agencies to ensure that limited capital and maintenance funds can be allocated and spent productively. The life-cycle costing for the complicated structures depend on an accurate significance of the deterioration processes. The deterioration models for chloride-induced corrosion in reinforced concrete members have been investigated by many researchers during the past decades (Vu and Stewart, 2000, Enright *et al*., 1998). Typically, the models are deterministic and use mean values as the input parameters. However, in recent years, there has been a trend towards incorporating the inherent uncertainties of the input parameters into service life calculation. There are three types of probabilistic models widely used today. These are regression models, Markov Chain models, and statistical computing techniques (Kirkpatrick, 2002). For reinforced concrete, the single most significant deterioration mechanism is corrosion of reinforcement due to the diffusion of chlorides from marine environment.

Corrosion-induced deterioration of reinforced concrete elements is a common and recurring problem for bridge structures located near coastal area or in marine environment. An evaluation procedure for determining a service life of the bridge structures generally involves two distinct phases. The first phase is referred to as an initiation phase in which reinforcing steel bars are protected by a thin oxide film developed during the cement hydration process. This protective action can however be destroyed due to a penetration of chloride ions into concrete. As the chloride concentration at reinforcing steel bars reaches a critical value, corrosion will occur, resulting in cracking and spalling of concrete cover, as well as a loss of steel section. The second phase is commonly called a propagation phase. Corrosion of reinforcement and spalling of the concrete cover will result in a reduction of load-carrying capacities and a serviceability problem. Consequently, without proper actions on the bridge maintenance program, the service life of bridge structures may be shorter than the value anticipated during the design process. A reasonable decision regarding maintenance, repair, and replacement of deteriorated elements must take into account the condition of structural elements, the extent of deterioration, the expected remaining service life, and the impact of alternative maintenance and repair options on service life of such elements. However, available publications do not provide reliable procedures for evaluating the existing bridge in Thailand with corrosion-damaged elements. Without such information, the process for selecting the optimum repair strategy becomes difficult.

In this research, a life cycle cost analysis of concrete bridge structures include several repair option, repair cost, and time to repair is determined and used to select the best maintenance plan based on the minimum cost. The approach was demonstrated using an example of deteriorated reinforced concrete bridges located in the southern part of Thailand.

OBJECTIVES

The objectives of this research are listed as following:

1. To develop probabilistic models for evaluating time to initial corrosion damage and service life of bridge structured subjected to chloride-induce corrosion.

2. To study life-cycle cost of concrete bridge structures which have deteriorated due to chloride-induced corrosion based on reliability approach.

3. To study the effect of corrosion rate and maintenance strategy on life cycle cost.

4. To determine the effective plan for repair and maintenance of reinforced concrete bridge structures which have deteriorated due to chloride induced corrosion.

Scope of research

The scope of this research study is listed as following:

1. Apparent chloride diffusion coefficients and apparent surface chloride contents were determined using field data collected from an example of deteriorated reinforced concrete bridges located in the southern part of Thailand.

2. The possible distributions of the chloride corrosion initiation concentration and corrosion rate were determined from the literatures.

3. This study utilized data from concrete bridge slab and pier to incorporate probabilistic considerations into a service life model. The service life was determined from load-carrying capacity of the members. The effects of cracking and loss of bond were ignored.

4. The optimization was based on minimizing the expected total life-cycle cost. The cost to consider in the study includes material, labor and construction cost.

LITERATURE REVIEW

Life-cycle cost for chloride-induced corrosion in reinforced concrete members has been investigated by many researchers over the past decades (Frangopol *et al*., 1997; Sancharoen *et al*., 2008). The overviews of research on this area are presented as below.

1. Deterioration Model

Chlorides may diffuse through the protective concrete cover. Corrosion is initiated once the chloride concentration at steel level exceeds a critical threshold value or if a flexure or bond crack width is sufficiently large to allow the direct ingress of chlorides. This is followed by corrosion propagation, causing losses in steel section and bond strength. A description of this deterioration process is shown in Figure 1.

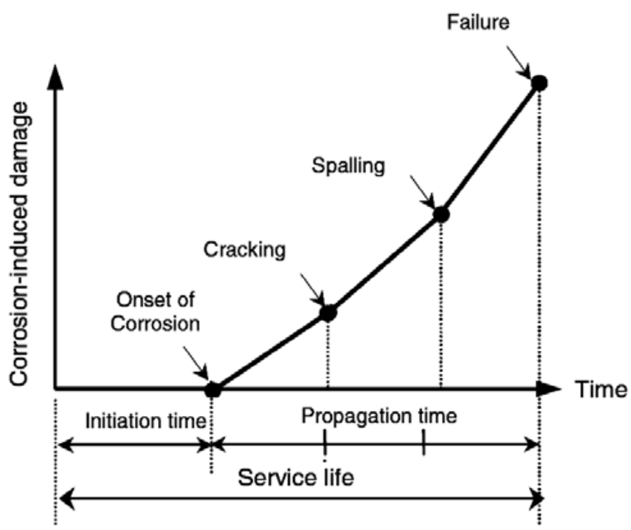


Figure 1 Service life model for chloride induced corrosion of reinforced concrete
Source: Morcous and Lounis (2005)

1.1 Time to corrosion initiation

An apparent diffusion process based on the Fick's second law is widely used to estimate the time for chloride ions to reach a critical value, initiating corrosion of reinforcing steel. The chloride concentration can be expressed as (Kirkpatrick, 2002):

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

Where	C	=	chloride concentration, kg/m^3
	x	=	concrete cover depth, cm
	t	=	time for diffusion, year
	D_C	=	apparent diffusion coefficient, mm2/year

Although the diffusion of chlorides is not restricted to one-dimensional movement, test results shows that the primary direction of transport is through the depth when the w/c ratio is low. Other work suggests that the true transport mechanism may be a combination of absorption, permeation under hydrostatic, pressure and ionic diffusion. Regardless of the true transport mechanism, the one dimensional diffusion model provides results that are reasonable estimates of measured data.

For the condition of constant surface chloride and a one-dimensional infinite depth, the solution to the Fick's law takes the following form (Kirkpatrick, 2002):

$$C_{(x,t)} = C_O(1 - erf\frac{x}{2\sqrt{D_C t}}) \tag{2}$$

Where	$C_{(x,t)}$	=	chloride concentration at depth and time, kg/m3

C_O	=	surface chloride concentration, kg/m3
D_C	=	apparent diffusion coefficient, mm2/year
t	=	time for diffusion, year
x	=	concrete cover depth, cm
erf	=	statistical error function

However, for a given bridge, the values of $C_{(x,t)}$, C_o, D_c and x are random variables, each of them has their own statistical distributions, means, and variances. A solution from Equation (2) for the diffusion time should include the probabilistic nature of the input variables. The time for corrosion damage to the end of functional service life is likely a random variable as well and depends on the corrosion rate, concrete cover depth, bar spacing, and steel diameter.

Monte Carlo simulation was employed in this study to determine the time to corrosion initiation. The simulation was a general class of repeated sampling methods where a desired response is determined by repeatedly solving a mathematical model using values randomly sampled from the assumed probability distributions of the input variables as shown in Figure 2 (Kirkpatrick, 2002).

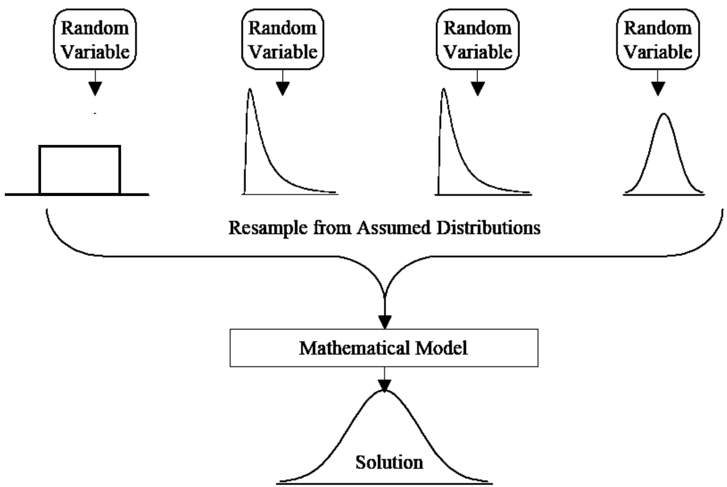


Figure 2 Parametric statistical simulation
Source: Kirkpatrick (2002)

1.2 Chloride corrosion initiation concentration

The concentration of chloride being necessary to initiate corrosion was not a fixed value, and significant variation exists between structures and within a structure. The chloride concentration being necessary to initiate corrosion in reinforced concrete may be influenced by many factors, including.

- Concrete mix proportions
- Cement type
- Tri-calcium aluminates' (C3A) content of the cement
- $\frac{W}{C}$ Ratio
- Temperature
- Relative humidity
- Source of chloride penetration

Based on the literature review conducted by Kirkpatrick (2002), a range of 0.6 to 1.2 kg/m3 (1.0 to 2.0 lb/cy) has been suggested as a conservative estimate for use in Cady-Weyers service life model (Weyers *et al.* 1993). Work on field structures in the United States reported that the range of chloride initiation is between 0.59 and 5.1 kg/m3 (1.0 to 8.5 lb/cy). Similar work on field structures in the UK reported a range of initiation values between 0.7 and 5.25 kg/m3 (1.2 to 8.75 lb/cy). A much higher range of 6.3 to 7.7 kg/m3 (10.6 to 13 lb/cy) was reported from Austrian bridges. A laboratory studied in Japan, using simulated bridge substructure components, reported the chloride initiation concentration as a lognormal distribution with an average of 3.07 kg/m3 (5.1 lb/cy) and a standard deviation of 1.26 kg/m3 (2.1 lb/cy) (Kirkpatrick, 2002).

1.3 Corrosion damage

After the initiation phase, corrosion will occur and result in a section loss of reinforcing steel bars. A reduction of bond is however not considered herein, as it appears generally to have a negligible effect on bridge reliability. During the propagation phase, a rebar diameter at any given time can be expressed as (Frangopol *et al.*, 1997):

$$D(t) = \begin{cases} D_i & t \le T_i \\ D_i - 2\lambda(t - T_i) & T_i < t \le T_i + (D_i / 2\lambda) \\ 0 & t > T_i + (D_i / 2\lambda) \end{cases} \tag{3}$$

Where	$D(t)$	=	diameter of the reinforcing steel at time t, mm
	D_i	=	initial bar diameter, mm
	T_i	=	time to diffusion, year
	λ	=	corrosion rate at a surface, mm/year

Corrosion rate can be expressed in term of current density as $\lambda \approx 0.0116 i_{\mathrm{corr}}$ (mm/yr) in the literature (Frangopol *et al.*, 1997).

1.4 Corrosion rate model

The corrosion rate is governed by the availability of water and oxygen at the steel surface, and so is probably a function of concrete quality and cover. Unfortunately, information on the effects of such phenomena is limited. The electrical resistance of the concrete is the governing factor when the ambient relative humidity is low. When relative humidity is high, the oxygen availability at the cathode is the controlling factor affecting corrosion rate.

For many locations in Australia, US, Europe and Asia shows the average relative humidity is over 70% (Vu *et al.*,2000) and so it is assumed herein that the corrosion rate is limited by the availability of oxygen at the steel surface. As such, the oxygen availability depends on concrete quality ($\frac{W}{C}$ ratio), cover, and environmental conditions (temperature and relative humidity). The oxygen diffusion rate is calculated from Fick's first law of diffusion with the oxygen diffusion coefficient obtained from Tuutti (1982). The conversion of the oxygen diffusion rate to the corrosion rate is made by considering the percentage of corrosion products and the molecular equations for corrosion reactions at the cathodic area. The results of such calculations are shown in Figure 3, which shows the effect of concrete quality and cover on the corrosion rate, for an ambient relative humidity of 75% and temperature of $20^{\circ}\mathrm{C}$. For this typical environmental condition, the influence of water-cement ratio and cover may be expressed empirically as Equation 4 (Liu and Weyers, 1998).

$$i_{\mathrm{corr}}(1) = \frac{37.8(1-\frac{w}{c})^{-1.64}}{\mathrm{cover}}(\mu\mathrm{A/cm^2}) \tag{4}$$

Where $i_{\mathrm{corr}}(1)$ = corrosion rate at the start of corrosion propagation and cover, cm

cover = concrete cover depth, cm

$\frac{W}{C}$ = water cement ratio

Data reported by Liu and Weyers (1998) is use to develop a relationship between time since initiation and the corrosion rate as shown in Figure 4, which is expressed empirically as

$$i_{\mathrm{corr}}(t_P) = i_{\mathrm{corr}}(1) \cdot 0.85 t_{\mathrm{P}}^{-0.29} \tag{5}$$

Where t_{P} = time since corrosion initiation , year

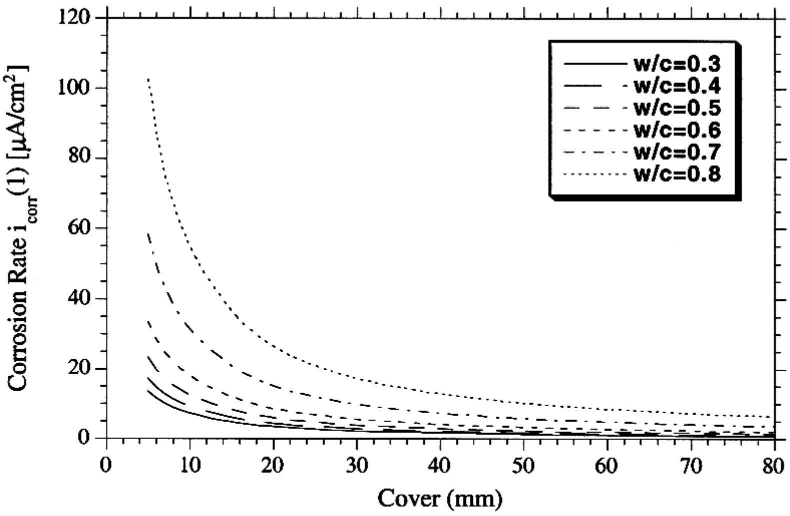


Figure 3 Influence of water-cement ratio and cover on corrosion rate
Source: Vu *et al.* (2000)

Much literature has assumed that the corrosion rate is constant during the propagation period, as shown in Tables 1 and 2. However, it is expected that the formation of rust products on the steel surface will reduce the diffusion of the iron ions from the steel surface. Also, the area ratio between the anode and cathode is reduced. This suggests that the corrosion rate will reduce with time, namely, rapidly during the first few years after initiation and then, slowly as it approaches a nearly uniform level.

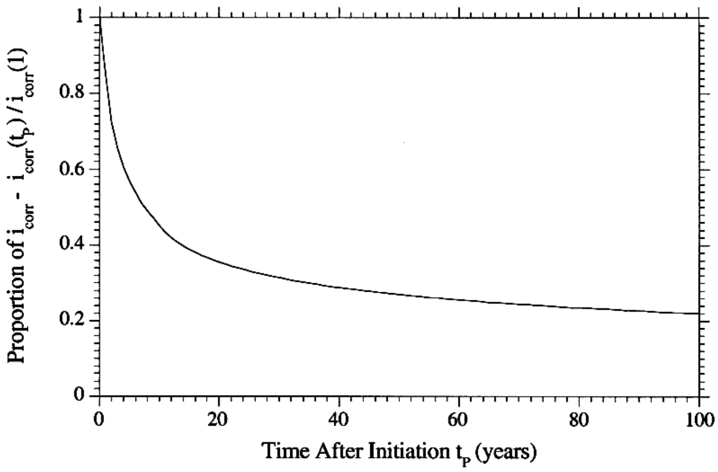


Figure 4 Effect of time on corrosion rate

Source: Vu *et al.* (2000)

Table 1 Summary of chloride concentration

C_o (%wt)		D_c (cm^2/yr)		x (cm)		Location	Refer
mean	**COV**	**mean**	**COV**	**mean**	**COV**		
0.100	0.050	0.320	0.050	3.810	0.050		
0.150	0.100	0.650	0.100	5.080	0.100		Michael P.Enright
0.200	0.015	1.290	0.150	6.350	0.200	USA	
0.300		1.940	0.200	7.620	0.300		
0.400		2.580					
						USA	Dan M. Frangopol
(1) 0.303	0.344	0.380	1.053	5.700	0.175	Colorado	
(2) 0.178	0.203	0.160	1.000	5.000	0.180	New Mexico	
(3) 0.444	0.150	0.180	0.500	5.300	0.132	Kentucky	NCHRP Report 88
(4) 0.344	0.292	0.340	0.412	5.600	0.143	Ohio	
(5) 0.477	0.195	0.720	0.931	6.400	0.078	Maryland	

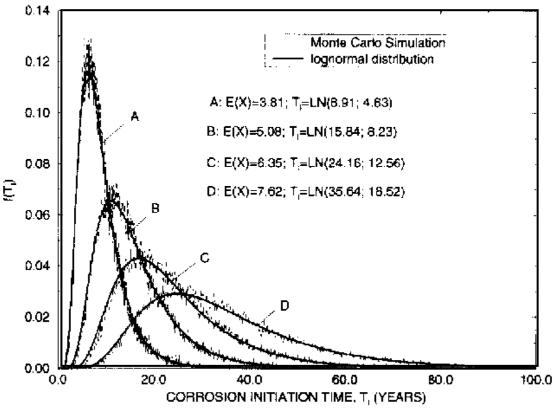
Source: Enright *et al.* (1998); Frangopol *et al.* (1998); NCHRP Report 88 (2006)

Table 2 Summary of corrosion rate

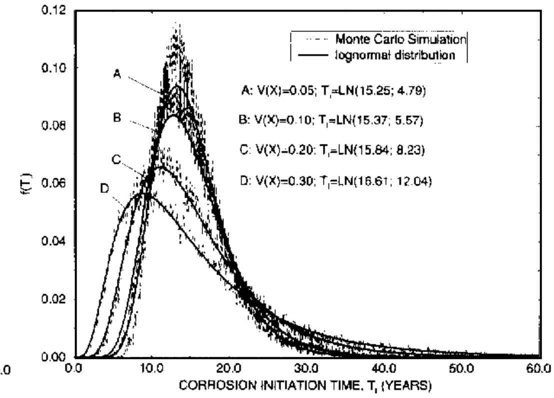
Corrosion rate (mm/yr)		Location	Refer
mean	**COV**		
0.013	0.100	USA	Michael P.Enright
0.025	0.200		
0.076	0.300		
0.127	0.400		
0.254	0.500		
0.064	0.391	USA	Dan M. Frangopol
0.089	0.281		
0.114	0.219		
2.573[(1)]	1.333	Colorado	NCHRP Report 88
0.083[(2)]	1.614	New Mexico	
0.118[(2)]	1.212		
0.187[(3)]	1.144	Kentucky	
0.172[(3)]	0.977		
0.162[(4)]	1.068	Ohio	
0.316[(4)]	0.741		
15.871[(5)]	0.738	Maryland	
0.343[(5)]	0.703		

Source: Enright *et al*. (1998); Frangopol *et al*. (1998); NCHRP Report 88 (2006)

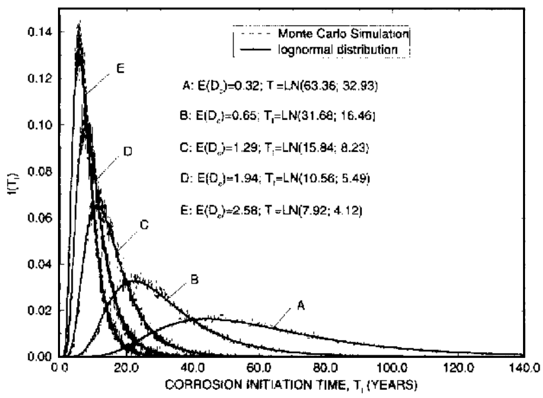
Enright *et al*. (1998) studied bridges subjected to environmental attack that can experience change in resistance which are time variant. Based on the experimental results, flexural strength loss in concrete bridge beams due to corrosion of steel reinforcement is considered. The sensitivity of the corrosion initiation time of steel reinforcement to changes in the main descript of diffusion related random variables is illustrated. The mean and standard deviation of the corrosion initiation time increase with an increase in the coefficient of variation of each of the diffusion related random variable for the range of values considered, as shown in Tables 1 and 2. An example problem given illustrates the effect of various variables including corrosion rate and corrosion initiation time on the time variant area of steel reinforcement and flexural strength of an existing reinforced concrete bridge beam, as presented in Figures 5 to 7.



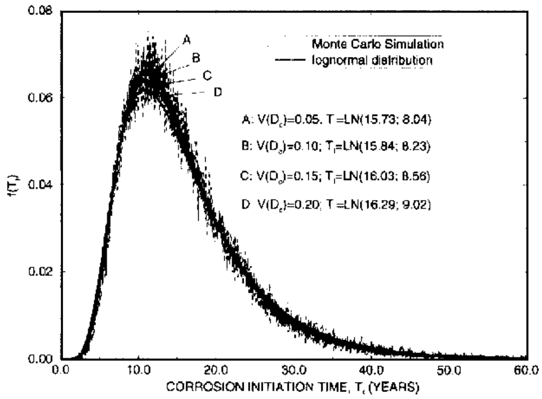
a.) Effect of mean cover depth



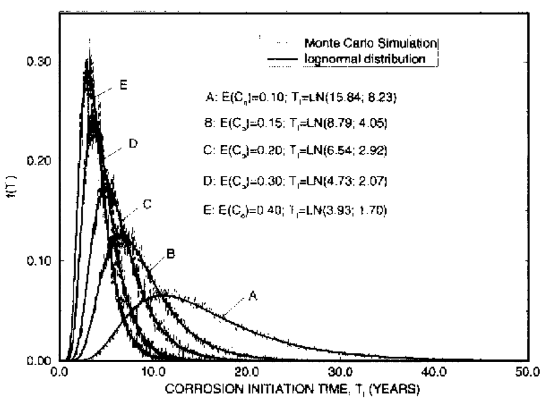
b.) Effect of COV cover depth



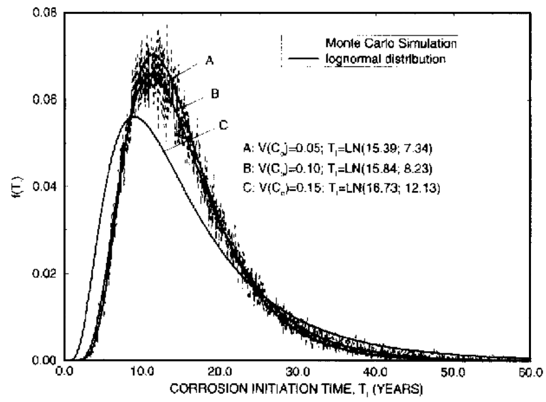
c.) Effect of mean diffusion coefficient



d.) Effect of COV diffusion coefficient



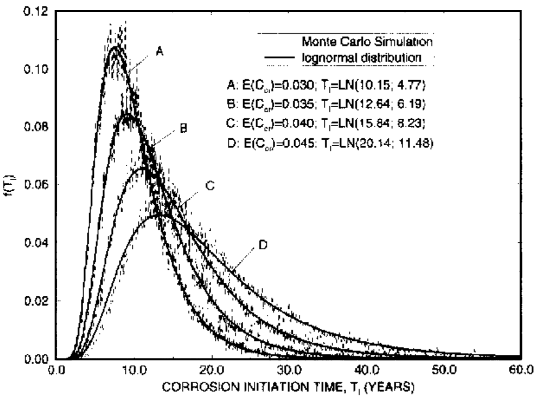
e.) Effect of mean surface chloride concentration



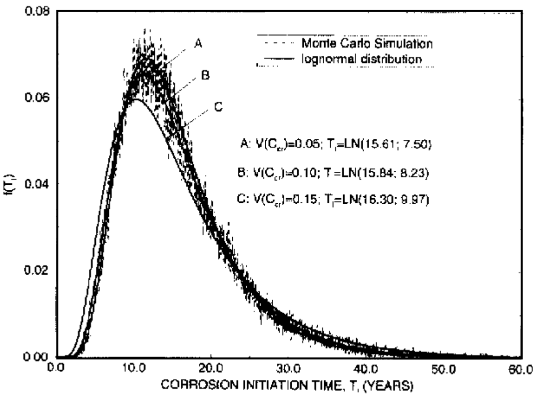
f.) Effect of COV surface chloride concentration

Figure 5 Effect of mean and coefficient of variation on corrosion initiation time

Source: Enright *et al*. (1998)



g.) Effect of mean critical chloride concentration



h.) Effect of COV critical chloride concentration

Figure 6 Effect of mean and coefficient of variation on corrosion initiation time.
Source: Enright *et al*. (1998)

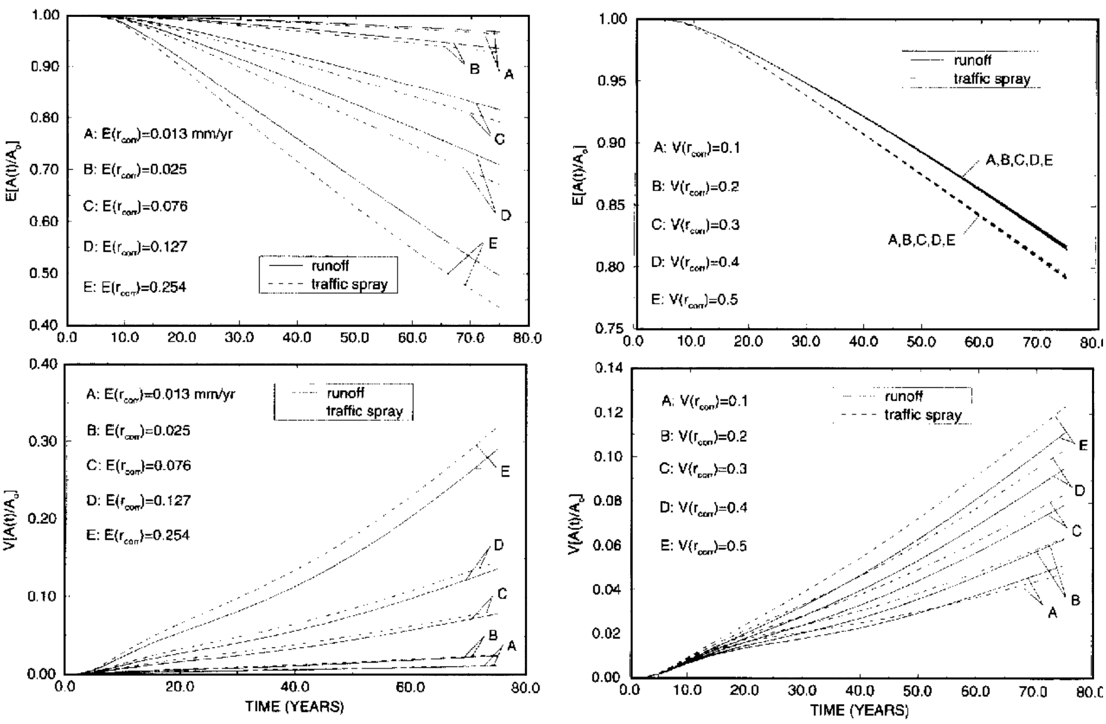


Figure 7 Effect of mean and coefficient of variation corrosion rate on normalized time variant area of bending reinforcement
Source: Enright *et al*. (1998)

Vu *et al.* (2000) studied the effect of durability to deterioration probabilistic model of structure. The reinforced concrete corrosion initiation, corrosion rate and time-variant load models are proposed. Three durability design specification are considered in a lifetime reliability analysis of a reinforced concrete slab bridge, as shown in Figures 8 and 9. It was found that the application of de-icing salts causes significant long-term deterioration and reduction in structural safety for poor durability design specifications, as shown in Figure 10. A reduced cover or increased water-cement ratio increases failure probabilities. When comparing to the case of no deterioration, it was observed also that the probability of failure only marginally increased for good durability design specifications. The approaches described herein are relevant to other physical infrastructure as well. Figure 11 shows the influence of moderate time-variant increasing in truck load and traffic volume for no deterioration.

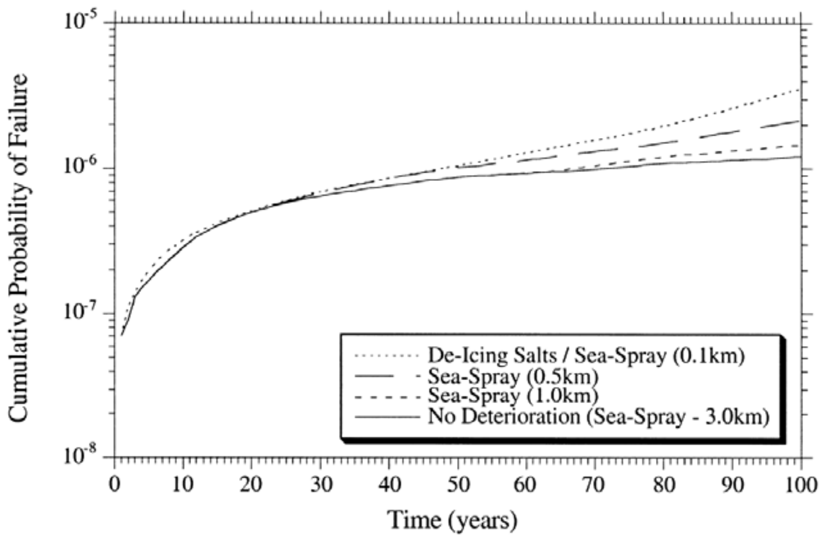


Figure 8 Time-dependent cumulative probabilities of failure, for fair durability design specification.

Source: Vu *et al.* (2000)

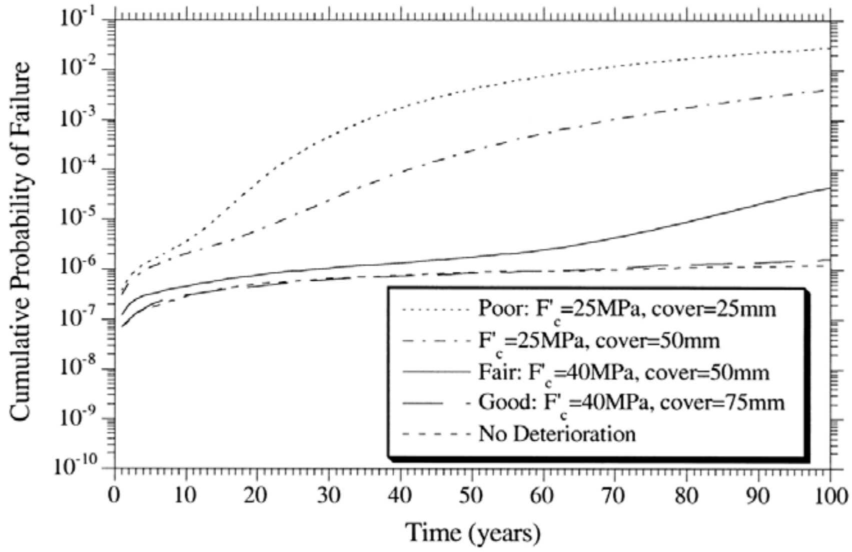


Figure 9 Time-dependent cumulative probabilities of failure for de-icing salts
Source: Vu *et al.* (2000)

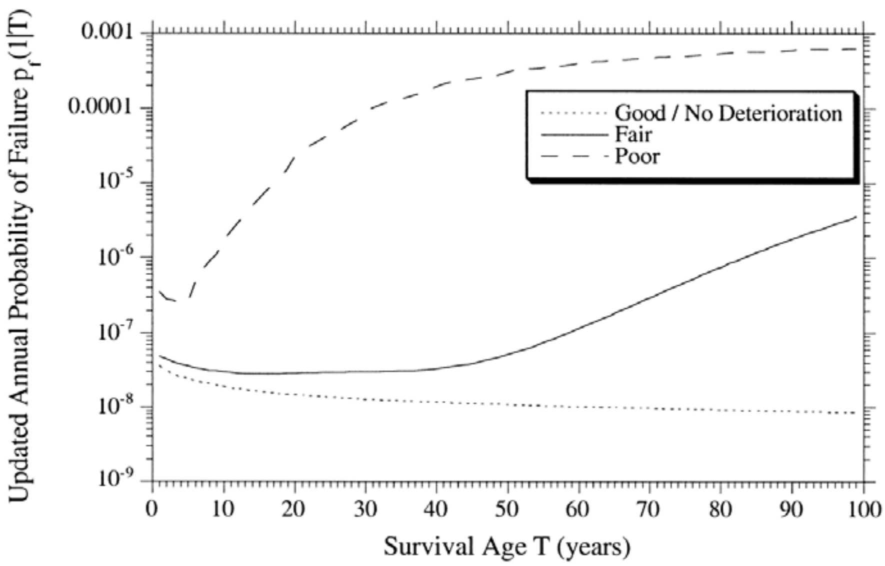


Figure 10 Updated annual probabilities of failure for de-icing salts
Source: Vu T *et al.* (2000)

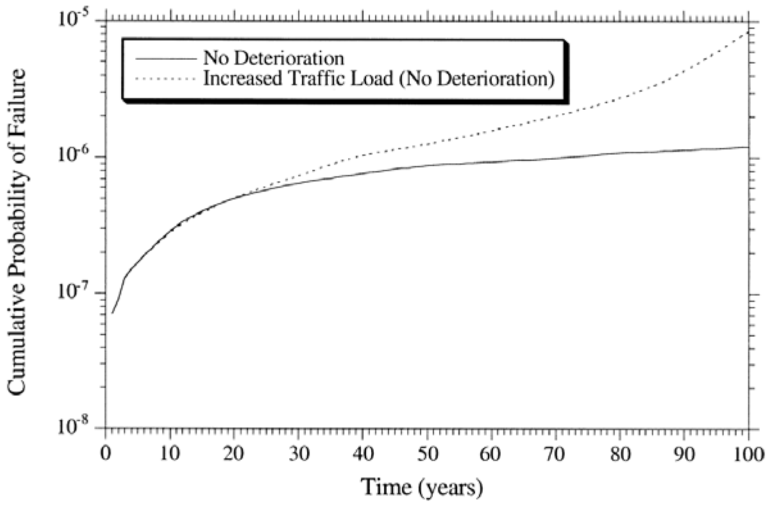


Figure 11 Time-dependent cumulative probabilities of failure for increased loads
Source: Vu *et al.* (2000)

Gelany (2001) studied the results of a laboratory investigation in which the applicability of Tafel plot and linear polarization techniques in short-term corrosion rate measurement of reinforcing bar in concrete had been evaluated. One hundred and four OPC and HPC concrete cylinders, fifty-two of each kind and cylinder with a single embedded reinforcing bar were subjected to various controlled conditions, and the corrosion rate of each specimen was monitored. The results indicate that sodium chloride concentration plays an effective role on the propagation of corrosion, *i.e.,* the higher the NaCl concentration, the higher the corrosion rate. The corrosion rate obtained from the linear polarization resistance technique is 20 to 30 percent higher than obtained from Tafel plot technique as shown in Table 3.

Kirkpatrick (2002) studied statistical model to determine the time to first repair and subsequent rehabilitation of concrete bridge decks exposed to chloride deiced salts that incorporates the statistical nature of factors affecting the corrosion process being developed. The result model expands on an existing deterministic model using statistical resampling techniques as shown in Table 4. Data collected for the time for corrosion deterioration after corrosion initiation can be readily incorporated into the model. Data for the surface chloride concentration, apparent diffusion coefficient and clear cover depth were collected from 10 bridge decks built in Virginia, as shown in Table 5.

Table 3 Value of polarization resistance technique versus Tafel plot technique

		Linear polarization resistance **Corrosion rate (mm/year)**	**Tafel plot** **Corrosion rate (mm/year)**
Cycle 1	E4(1)	0.005	0.043
	E4(2)	0.011	0.091
	E4(3)	0.012	0.095
Cycle 2	E4(1)	0.008	0.069
	E4(2)	0.680	0.599
	E4(3)	0.910	0.7
Cycle 3	E4(1)	0.096	0.085
	E4(2)	0.740	0.693
	E4(3)	1.040	0.802
Cycle 4	E4(1)	0.110	0.0949
	E4(2)	0.850	0.709
	E4(3)	1.140	0.914
Cycle 5	E4(1)	0.116	0.097
	E4(2)	0.900	0.75
	E4(3)	1.200	0.96
Cycle 6	E4(1)	0.144	0.12
	E4(2)	0.940	0.78
	E4(3)	1.230	0.98
Cycle 7	E4(1)	0.160	0.134
	E4(2)	1.080	0.9
	E4(3)	1.400	1.1

Source: Gelany (2001)

Table 4 Time to first repair and rehabilitation (in years)

$C_{(x,t)}$ **from 0.60 to 1.20 kg/m^2**						
Structure	**Parametric bootstrap**			**Simple bootstrap**		
number	**%corroded**	**2.50%**	**12%**	**%corroded**	**2.50%**	**12%**
1001	100	10	13	100	11	13
1004	100	23	31	100	23	30
1004	100	33	46	100	33	47
1015	100	28	48	100	28	49
1019	99	30	47	100	33	45
1136	96	31	56	91	34	52
2021	27	-	-	27	-	-
2262	19	-	-	18	-	-
6037	7	-	-	8	-	-
6128	0	-	-	0	-	-

Source: Kirkpatrick (2002)

Table 5 Summary of bridge decks

District		Structure number	Year built	Age at sampling (years)	Reinforcent type
7	Culpeper	1001	1992	7	ECR
6	Fredericksburg	1004	1993	6	ECR
3	Lynchburg	1004	1983	16	ECR
2	Salem	1015	1987	12	ECR
7	Culpeper	1019	1990	9	ECR
1	Bristol	1136	1995	4	ECR
5	Suffolk	2021	1981	18	ECR
9	Northerm Virginia	2262	1985	14	ECR
1	Bristol	6037	1983	16	Bare Steel
2	Salem	6128	1981	18	Bare Steel

Source: Kirkpatrick (2002)

Daigle *et al.* (2004) presents an evaluation of the effectiveness and benefits of using high-performance concrete (HPC) containing corrosion inhibitors in the construction and repair of concrete bridges. Numerical models for predicting the early-age and in-service performance of HPC bridge structures are shown in Figure 12. The main parameters of these models, originally conceived for normal concrete, are revised and adapted for using with HPC containing corrosion inhibiting systems. The proposed models focus on the problems of early-age cracking, chloride ingress into concrete and corrosion rate of the conventional reinforcing steel, as shown in Figures 13 and 14. A case study of reinforced concrete (RC) barrier walls was used to illustrate the predictive capabilities of the models and the benefits of using HPC in extending the service life of bridge structures.

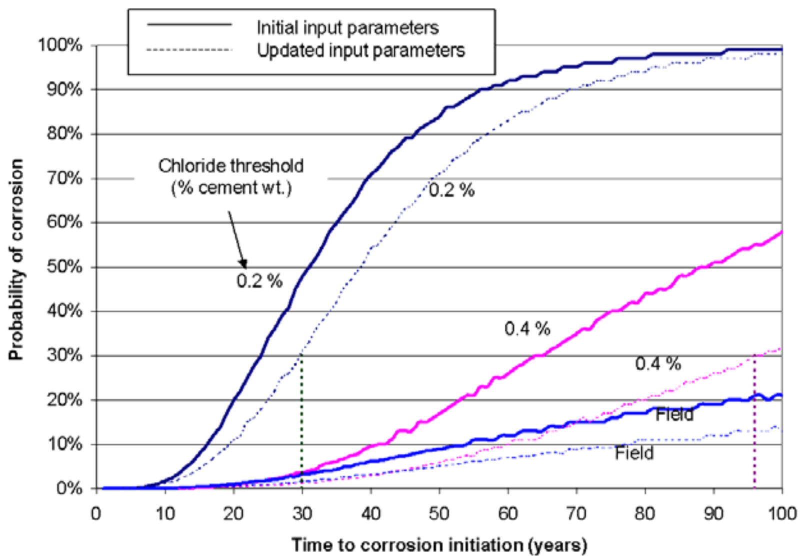


Figure 12 Prediction of time-dependent probability of corrosion

Source: Daigle et al. (2004)

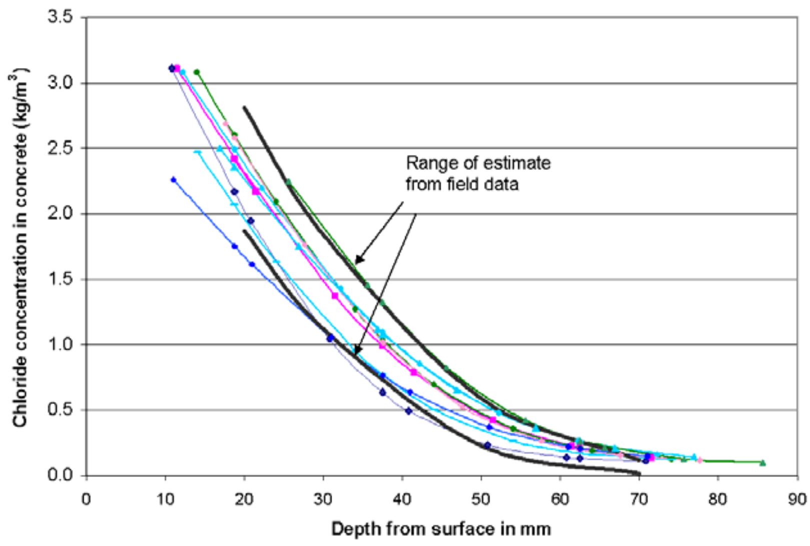


Figure 13 Chloride profiles in the barrier wall after 5 years parameter from field measurements

Source: Daigle *et al.* (2004)

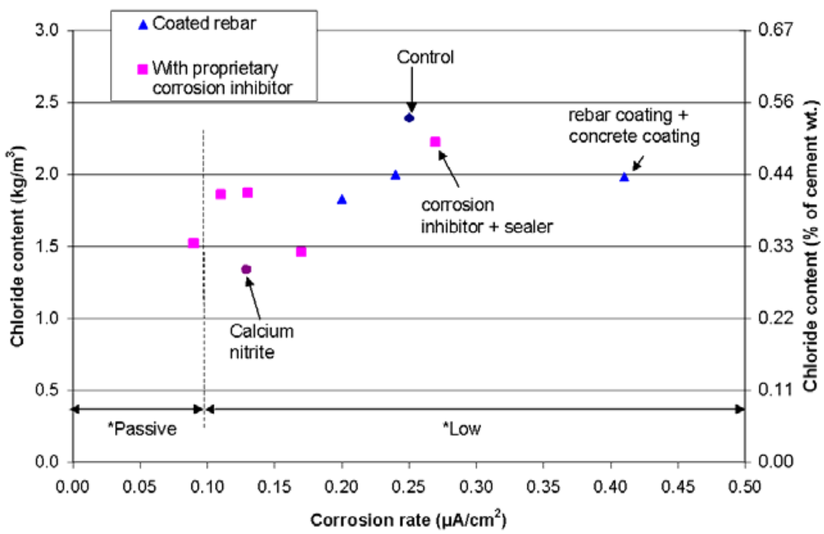


Figure 14 Corrosion rate after five years on special reinforcing bars installed at 25 mm depth.

Source: Daigle *et al.* (2004)

Juleang *et al*. (2008) developed probabilistic models for predicting the maintenance free service life of reinforced concrete structures in chloride environment in Thailand. The variations of chloride diffusion coefficient and surface chloride content of concrete were taken into account in this model. In addition, the effect of water to binder ratio on the chloride diffusion and surface chloride content was investigated by using the Fick's second law of diffusion. The results indicated that the chloride diffusion coefficient of concrete and the surface chloride content of concrete were different according to types of the structure, water to binder ratio and exposure period in chloride environment.

2. Limit state function

A limit state is defined as a boundary between desired and undesired performance of a structure. A mathematical formulation of the limit state is well known as a limit state function. The limit state function might include ultimate capacity, serviceability, loss of section or any other ranges of criteria. The ultimate limit states are mostly related to loss of load-carrying capacity. Examples of the criteria for the ultimate limit states are torsion, flexural and shear capacities. The serviceability limit states are mostly related to gradual deterioration, user discomfort, and maintenance costs.

The structural reliability theory provides a rational analysis procedure that can be used to calculate a probability of failure in a structure subjected to loads. In this study, the structural safety of a structural component is evaluated using its failure probability defined as follows

$$\begin{aligned} \text{Failure Probability } P_f &= \text{Probability[Resistance- Load Effect} < 0 \text{]} \\ &= \text{probability[R-Q<0]} \end{aligned} \tag{6}$$

Where P_f = Probability of failure

R = Resistance of load carrying capacity

Q = Load demand on the component

For example, the load effect can be bending moment for a beam section and the resistance is the beam section's moment capacity. The resistance and load effect in Equation 6 are modeled as random variables because both of them possess an amount of uncertainty. In general, the uncertainties associated with the resistance are due to material properties and the production and preparation process, construction quality, etc. The uncertainty associated with load effect is related to truck weight, truck type, traffic volume, etc. Note that the failure probability in Equation 6 refers to a load effect in a structural component. Namely, this definition can be applied to a variety of load effects, such as moment, shear, or even possibly displacement if this serviceability is an issue. It also can be applied to a variety of bridge structural components, such as beams, slabs, piers, etc.

The reliability index is used to represent a level of structural safety. A probability of failure (P_f) can be defined in terms of the reliability index, as shown in Equation 7. The corresponding probability of survival of a structural element or system is defined as Equation 8.

$$P_f = \Phi(-\beta) \tag{7}$$

$$P_s = 1-P_f \tag{8}$$

Where Φ = the cumulative density function of standard normal distribution

P_f = probability of failure

P_s = probability of survival

β = reliability index

The relationship between a selected value of reliability index (β) and a probability of failure (P_f) is shown in Table 6

Table 6 Relationship between reliability index and probability of failure

Reliability Index	Probability of Failure
0	0.5
1	0.159
2	0.0228
3	0.00135
4	0.0000317
5	0.000000287
6	0.000000000987

Amatayakul (1988) studied reinforced concrete members using ultimate strength theory. The capacity reduction factors are required to control some variations of materials, size, workmanship and design method. In this study, the number of variables, which would directly affect the strength capacity of reinforced concrete members, were collected from several construction sites in Bangkok Metropolitan area. Monte Carlo simulation was employed to determine the reliability index. The major variables affecting the strength of reinforced concrete members which have actual yield strength of reinforcing steel is greater then 44% for SR24 while yield strength of reinforcing steel is 31% and 15% , higher than the specified value for SD30 and SD40 respectively. For concrete with specified compressive strength higher than 200 kg/cm^2, the average value was about 20 kg/cm^2 less than the specified value. The member cross-sections tend to about 0.2-0.5 cm which are 0.1-1.8 cm less than the specified values.

Nowak (1995) reviews the code development procedures used for the new load and resistance factor design (LRFD) bridge code. The code is based on a probability-based approach. Structural performance is measured in terms of the reliability index or the probability of failure. Load and resistance factors are derived so that the reliability of bridges designed using the proposed provisions will be at predefined target level. The paper describes the calibration procedure. A new load model is proposed, which provides a consistent safety margin for a wide spectrum of spans. The current design live load is based on a HS-20 truck, lane loading, or military loading, as shown in Figure 15. For a single lane, the bias factor (rations of mean maximum 75 year moment/shear and HS-20 moment/shear) are ploted versus the span length in Figure 16. The dynamic load model takes into account the effect of road roughness, bridge dynamics, and vehicle dynamics. Statistical models of resistance (load-carrying capacity) are summarized for noncomposite steel, composite steel, reinforced concrete, and prestressed concrete.

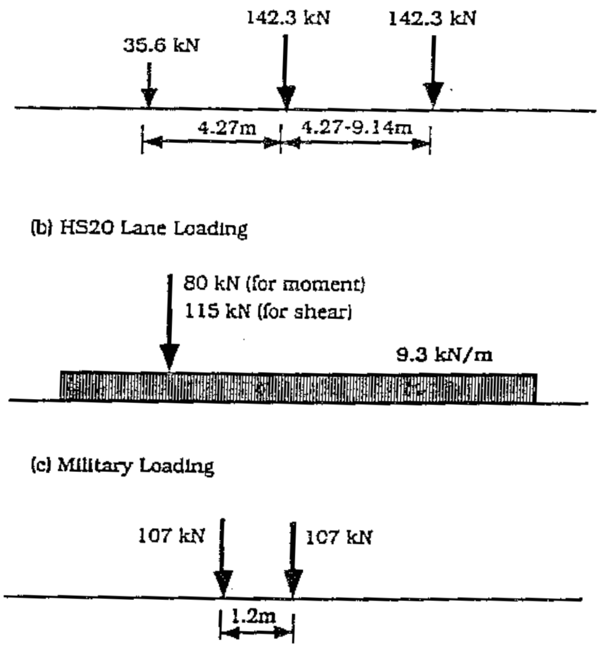


Figure 15 Nominal live load HS-20 truck and lane load (Standard 1992)
Source: Nowak (1995)

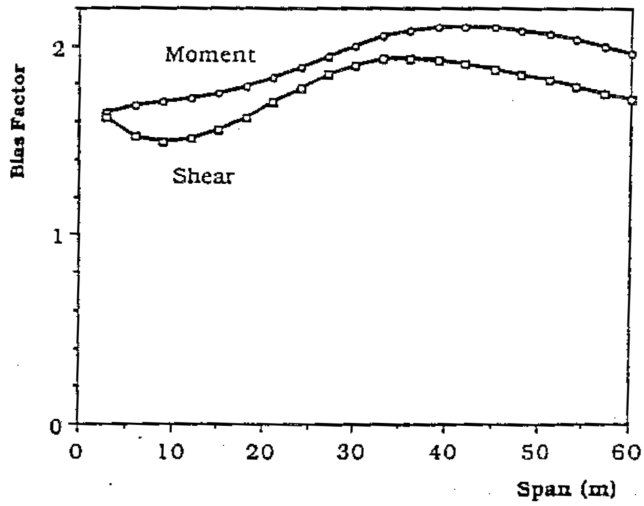


Figure 16 Bias factor for moment and shear AASHTO(1992)
Source: Nowak (1995)

NCHRP Report 88 (2006) providing the primary product of the effort was the development of manual titled “Manual on Service Life Prediction of Corrosion-Damaged Reinforced Concrete Bridge Superstructure Elements”. This manual provides a protocol for assessing the condition of reinforced concrete bridge superstructure elements subjected to corrosion-induced deterioration, predicting the remaining service life of such elements used to develope service life model, and quantifying service life extension for such elements expected from alternative maintenance and repair options. The report documents in which the data is utilized in the development and validation of the service life model is presented in the manual.

3. Procedures for solving limit state function

3.1 Second Moment Methods

The second-moment methods can be used to simplify the probability density functions. The second-moment formulation was developed as a practical approach representing the uncertainty only in terms of mean and variance. These methods focus on the determination of the design point, which is the point on the limit-state surface with the shortest distance from origin in the standard normal space. The methods are simply used and powerful for solving a wide range of practical problems. The second moment methods provide exact solutions only when the parameters in a limit state function are all defined as normal or lognormal distribution. Otherwise, only approximate solutions will be obtained.

A version of the reliability index is defined as the inverse of the coefficient of variation. The reliability index which is the shortest distance from the origin of reduced variables to the failure surface is illustrated in Figure 17. This definition was introduced by Hasofer and Lind (1974) from the following formula:

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \tag{9}$$

Where μ_R = mean resistance

μ_Q = mean load

σ_R = standard deviation of resistance

σ_Q = standard deviation of load

β = reliability index

When R and Q are uncorrelated for normally distributed random variables R and Q

However, a limit state function is generally not linear. Thus, the limit state function will not be normally distributed, although all random variables are normally distributed. A Taylor series expansion of the performance function about the mean value is given by Equation 10.

$$Z \approx G(\mu_1,\mu_2,....,\mu_n)+\sum_{i=1}^{n}(x_i-\mu_i)\left.\frac{\partial G}{\partial x_i}\right|_{x_i=\mu_i} \tag{10}$$

While the mean and variance can be determined from Equations 11 and 12.

$$\mu_Z \approx G(\mu_1,\mu_2,....,\mu_n) \tag{11}$$

$$\sigma_Z^2 \approx \sum_{i=1}^{n}\sum_{j=1}^{n}\left.\frac{\partial G}{\partial x_i}\right|_{x_i=\mu_i}\left.\frac{\partial G}{\partial x_j}\right|_{x_j=\mu_j} \mathrm{cov}(x_i,x_j) \tag{12}$$

If the random variables are independent, $\mathrm{cov}(x_i,x_j)$ will equal to zero. The variance in Equation 13 can be more simplified, as expressed in Equation 13.

$$\sigma_Z^2 \approx \sum_{i=1}^{n}\left[\left.\frac{\partial G}{\partial x_i}\right|_{x_i=\mu_i}\right]^2 \mathrm{Var}(x_i) \tag{13}$$

The reliability index can be calculated by taking the ratio of mean and standard deviation of Z as in Equation 14.

$$\beta = \frac{\mu_Z}{\sigma_Z} \text{ and } P_f = \Phi(-\beta) \tag{14}$$

Because the first-order Taylor series expansion is used in the approximation, the above procedure is so called the first-order second moment method. This method has some deficiencies that should be noted here. The distribution information of random variables is not considered in this method. In addition, the reliability index is not consistent under different formulations of the same limit state function. The lack of invariance problem can be overcome by Hasofer-Lind transformation.

Unlike the first-order second-moment method, the Hasofer-Lind transformation calculates a reliability index based on geometry of the limit state function. To apply this method, all random variables have to be uncorrelated and normally distributed. However, when the variables are correlated, an intermediate step is required to obtain the uncorrelated random variables. The procedure used to obtain these variables is essentially an eigen-value problem.

To apply the Hasofer-Lind transformation, all variables have to be first transformed to standard normal variables (u_i), by using Equation 15. As a result of this transformation, u_i will have zero mean and standard deviation equal to 1. The limit state function is also transformed and is given by $g(u_i)$.

$$u_i = \frac{x_i - \mu_i}{\sigma_i} \tag{15}$$

The reliability index can be calculated from a minimum distance from the origin of the axes of transformed coordinate system to the limit state surface ($g(u_i) = 0$), as shown in Figure 17 and given in Equation 16. The particular point satisfying Equation 16 is called a design point or checking point. With this definition, the reliability index is invariant because regardless of the form of the limit state equation, its geometric shape and distance from the origin remain constant.

$$\beta = \min(\sum_{i=1}^{n} u_i^2)^{1/2} \tag{16}$$

From the geometry of surfaces, the reliability index in Equation 17 can be determined from Equation 17 to 19. An iterative procedure is required to compute the reliability index and proceeds until a convergent value of the reliability index is obtained.

$$\beta = -\frac{\sum_{i=1}^{n} u_i^* \left(\frac{\partial g}{\partial u_i}\right)\Big|_{\underline{u}^*}}{\sqrt{\sum_{i=1}^{n} \left(\frac{\partial g}{\partial u_i}\right)^2}\Bigg|_{\underline{u}^*}} \tag{17}$$

Where $\underline{u}^*$ is the design point $(u_1^*, u_2^*, ..., u_n^*)$.

$$u_i^* = -\alpha_i \beta \tag{18}$$

$$\alpha_i = \frac{\left(\frac{\partial g}{\partial u_i}\right)\Big|_{\underline{u}^*}}{\sqrt{\sum_{i=1}^{n} \left(\frac{\partial g}{\partial u_i}\right)^2}\Bigg|_{\underline{u}^*}} \tag{19}$$

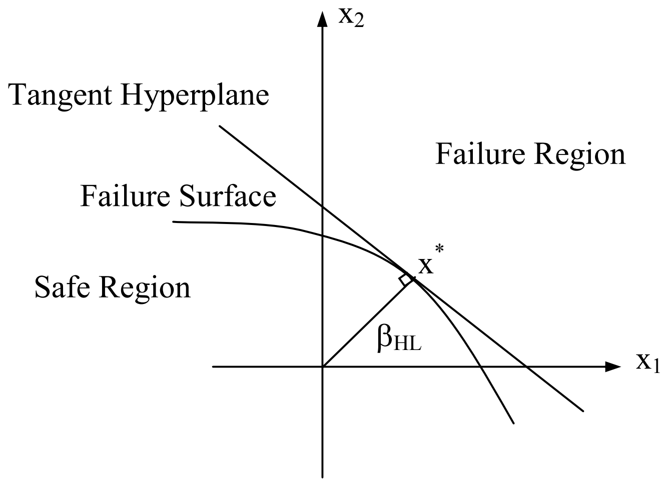


Figure 17 Description of Hasofer-Lind Reliability Index

Source: Hasofer and Lind (1974)

3.2 Rackwitz - Fiessler Method

Rackwitz-Fiessler suggested one approach improving the accuracy of probability of failure as an approximation to the true underlying probability density function of the system response. When using probability of failure as an approximation, it is inherently assumed that the system response can be accurately represented by a linear combination of Gaussian distributed random variables. When the true underlying distributions are significantly non-Gaussian, the approximation can have significant error, particularly when investigating regions in the tails of the distributions. The Rackwitz-Fiessler method, also referred as the 'equivalent normal' method, is based on developing a better Gaussian approximation to the true probability density functions in the area of interest within the design space(David G. Robinson, 1998).

A value of the design point $\left(x_i^*\right)$ has to be guessed first. Then, all non-normal variables have to be transformed into equivalent normal variables. The equivalent normal mean $\left(\mu_i^N\right)$ and standard deviation $\left(\sigma_i^N\right)$ of the guessed design point are determined based on the constraints provided in Equation 20 and 21.

$$f_x^{/}\left(x_i^*\right) = f_x\left(x_i^*\right) \tag{20}$$

$$F_x^{/}\left(x_i^*\right) = F_x\left(x_i^*\right) \tag{21}$$

Where $f_x^{/}$ = normal probability density function

$F_x^{/}$ = normal cumulative density function

f_x = probability density function of variable x_i.

F_x = cumulative density function of variable x_i.

x_i = value of the design point

The equivalent normal variables can be calculated from Equations 22 and 23.

$$\mu_i^N = x_i^* - \sigma_i^N \Phi^{-1}\left[F_{x_i}\left(x_i^*\right)\right] \tag{22}$$

$$\sigma_i^N = \frac{\varphi\left\{\Phi^{-1}\left[F_{x_i}\left(x_i^*\right)\right]\right\}}{f_{x_i}\left(x_i^*\right)} \tag{23}$$

Where φ = probability density functions of the standard normal distribution

Φ = cumulative density functions of the standard normal distribution.

μ_i^N = mean of equivalent normal distribution parameters

σ_i^N = standard deviation of equivalent normal distribution parameters.

These equivalent normal mean and standard deviation values need to be updated in each iteration. Then, the design point, u_i^*, in the transformed coordinate system can be obtained from Equation 24.

$$u_i^* = \frac{x_i^* - \mu_i^N}{\sigma_i^N} \tag{24}$$

Where u_i^* = the design point in the transformed coordinate system

The design point of the next design point, $u_{(i+1)}^*$, can be determined from Equation 25. This equation bases on the first-order Taylor series expansion about u_i^*.

$$\underline{u}_{(i+1)}^* = \frac{1}{\left|\nabla G\left(\underline{u}_i^*\right)\right|^2}\left[\left(\nabla G\left(\underline{u}_i^*\right)\right)^T \underline{u}_i^* - G\left(\underline{u}_i^*\right)\right]\nabla G\left(\underline{u}_i^*\right) \tag{25}$$

Where

$$\left(\nabla G\left(\underline{u}_i^*\right)\right)^T = \left[\frac{\partial G}{\partial u_1} \quad \frac{\partial G}{\partial u_2} \quad \ldots \quad \frac{\partial G}{\partial u_n}\right]_{\underline{u}_i^*} \tag{26}$$

$$\left(u_i^*\right)^T = \left[u_{1(i)} \quad u_{2(i)} \quad \ldots \quad u_{n(i)}\right] \tag{27}$$

The iterative procedure is used in this method. The algorithm iterations proceed until:

$$\left|u_{(i+1)}^* - u_i^*\right| \leq \delta \tag{28}$$

$$\left| \; G\left(u_{(i+1)}^*\right) \; \right| \leq \varepsilon \tag{29}$$

Where δ, ε = specified tolerance values.

A convergence of the Rackwitz-Fiessler method is often slower than the second-moment methods.

3.3 Monte Carlo Simulation

Monte Carlo simulation is a powerful tool in solving integration problems when random variables are related through nonlinear equations. It is very useful for evaluating limit states especially when these are implicit functions of the random variables. The Monte Carlo Simulation is based on the fact that sampling averages will be stable as a sample size increases. Therefore, the method involves randomly sampling to simulate artificially a large number of experiments.

The first step of this method is to define a problem in terms of random variables with known distributions. These random variables will then be generated. The corresponding value of safety margin can be determined from the simulation. The procedure will be repeated many times. With a suitable large number of generations, a probability of failure can be approximated by Equation 30.

$$P_f \approx \frac{n(G \leq 0)}{N} \qquad (30)$$

Where $n(G \leq 0)$ is a number of trials for which $G \leq 0$, and N is a total number of trials. Obviously, the accuracy of the probability of failure (P_f) depends on the number of simulations. Although the Monte Carlo simulation can be employed to solve a complex limit state function, it tends to be expensive when the limit state function is related to a very large number of random variables.

4. Optimization process

4.1 Fault Tree Decision

The Fault Tree Decision model provides a systematic deterministic value of structuring and evaluating the maintenance method possibilities and uncertain maintenance environment. It defines clearly and precisely the total environment. In this study, the Fault Tree Decision is used as a model to represent all possible events associated with no maintenance, minor repair, major repair or replacement action.

To construct an tree diagram, it is recognized that a decision to either repair or not repair needs to be made after every evaluation. Repair decisions made after every new evaluation are influenced by decisions made in the past. For example, the decision whether or not to repair after the second evaluation will be influenced by whether or not the structure was repaired after the first evaluation as presented in Figure 18. As the number of inspection, m, increases, the number of branches, 2m, in the tree diagram increases much faster.

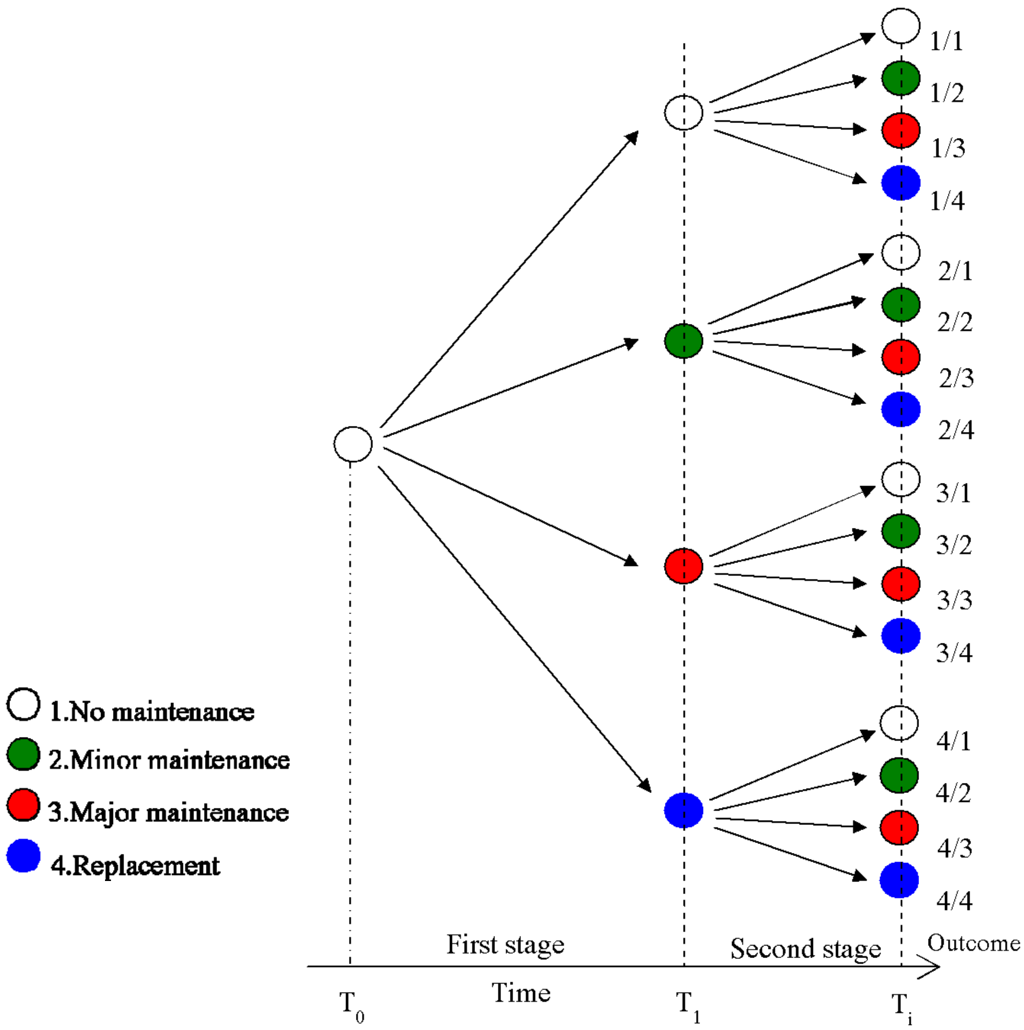


Figure 18 Tree diagram to identify the possible combinations of outcomes.

4.2 Life-Cycle Cost Optimization

The life time also called life cycle cost target life time reliability, inspection interval, and quality of repair must all be considered when optimizing the inspection/repair strategy of structural systems. There is a trade-off between a higher reliability and minimum expected total cost. The goal of an optimal repair strategy is to minimize the lifetime cost of a given structure while ensuring that the structure maintains an acceptable reliability level throughout its expected service life.

The assumptions used to compute the optimal life-cycle cost solution are as follows:

1. The deterioration mechanism considered is associated with general corrosion.

2. The loading, material properties, and time-dependent limit state function that describes the moment capacity of a reinforced concrete bridge subjected to corrosion.

3. The time value of money was considered using a constant interest rate over time. The discount rate (r), is used to convert the future cost to present cost.

4. If damage was found then a repair action will follow. If the damage is not found then the repair action will be postponed until the next evaluation.

The lifetime maintenance program of structural systems can be formulated as a cost optimization problem, which is subject to constraint on failure probability and times of inspection/repairs.

$$\min C_T = C_{INIT} + C_{INSP} + C_{REP} + C_{FAIL} \tag{31}$$

Subject to

$$P_f(75) \le P_f^0 \tag{32}$$

Where

$$t_{R_i} - t_{R_i - 1} \ge t_{\min} \qquad i = 1, n \tag{33}$$

$$C_{INSP} = \sum_{i=1}^{n} \frac{C_{IN_i}}{(1+r)^{t_{R_i}}}, \; C_{REP} = \sum_{i=1}^{n} \frac{C_{R_i}}{(1+r)^{t_{R_i}}}, \; C_{FAIL} = \sum_{i=1}^{n} \frac{C_F P_f(75)}{(1+r)^{75}} \tag{34}$$

Where

C_T	=	total cost
C_{INIT}	=	initial cost
C_{INSP}	=	cost of all inspections
C_{REP}	=	cost of all repairs
C_{FAIL}	=	cost of failure
i	=	number of inspection/repair
n	=	total number of inspection/repairs
r	=	discount rate of money
t_{R_i}	=	time of inspection/repair
C_{IN_i}	=	cost of inspection at i
C_{R_i}	=	cost of repair at i
C_F	=	failure cost coefficient
$P_f(75)$	=	probability of failure of structure member
P_f^0	=	target lifetime failure probability
$t_{\min}$	=	minimum time between inspections

Frangopol *et al.* (1997) proposed a life-cycle cost model to optimize the lifetime inspection and repair of concrete bridge structures which deteriorate over time which is introduced and illustrated through numerical examples. The optimization is based on minimizing the expected total life-cycle cost while maintaining allowable lifetime reliability for the structure. The method incorporates: (a) the quality of inspection techniques with different detection capabilities; (b) all repair possibilities based on an event tree; (c) the effects of aging, deterioration, and subsequent repair on structural reliability; and (d) the time value of money. The overall cost to be minimized includes the initial cost and the costs of preventive maintenance, inspection, repair, and failure. The methodology is illustrated using the reinforced concrete T-girders from a highway bridge. To demonstrate the effect of corrosion rate, three corrosion rates = 0.064cm/year 0.089cm/year and 0.0114 cm/year, were considered as shown in Figure 19.

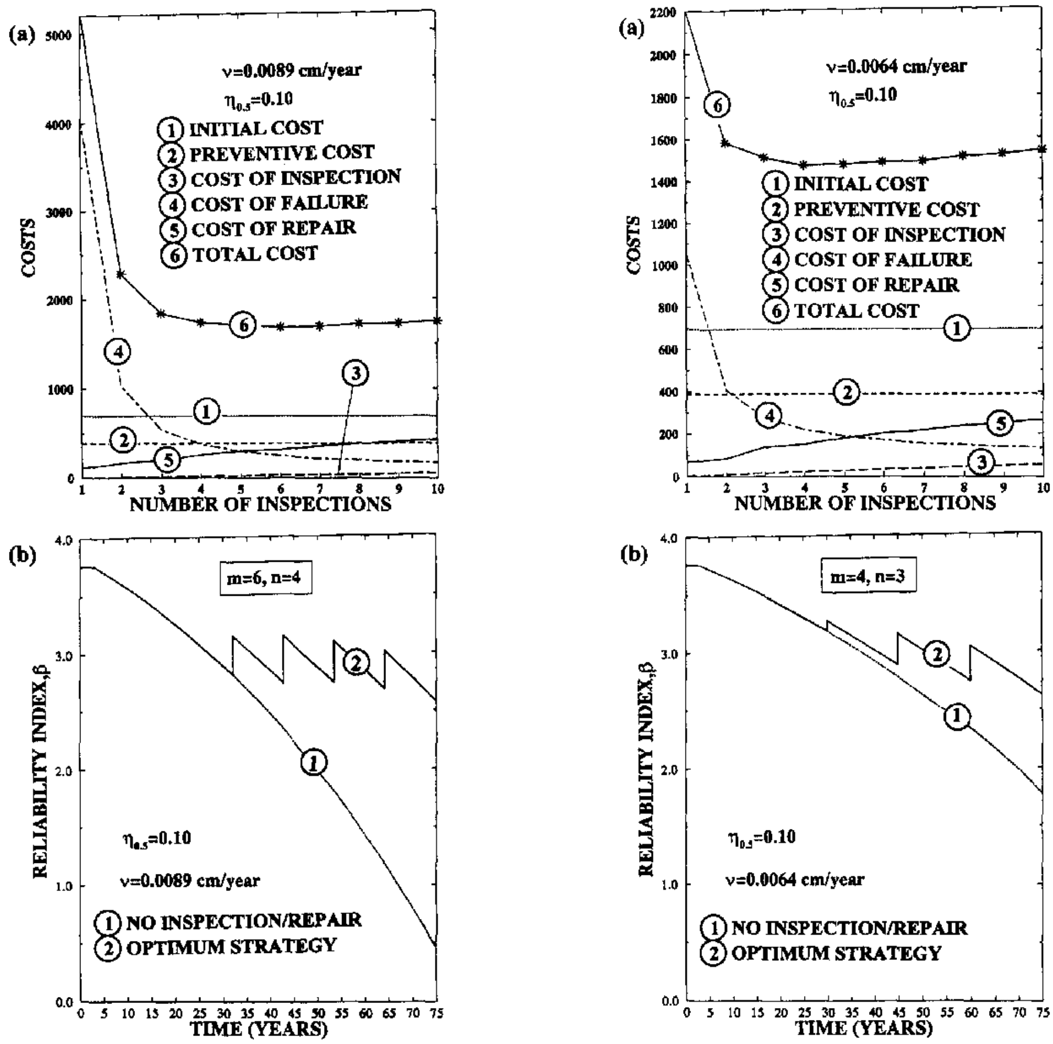


Figure 19 (a) Cost as function of number of uniform interval inspections

(b) Optimum inspection/repair strategy and inspection method

Source: Frangopol *et al.* (1997)

Sancharoen *et al*. (2008) proposed method of planning maintenance program for deteriorated reinforced concrete structure based on actual inspection result and prediction result of future condition of reinforced concrete structure. The reinforced concrete structures deteriorated by chloride attack in Thailand were inspected to evaluate their current conditions, such as covering depth, diffusion coefficient of chloride ion, surface chloride content, and compressive strength. Background information of each structure is shown in Table 7. As a result, only actual variations of the data are shown in Table 8. Very high variations are observed in both surface chloride content and chloride diffusion coefficient. The result shows that the proposed method can allow plan of the maintenance program for deteriorated reinforced concrete structure. Combinations of different repairing methods is also considered such as combination of patching with surface coating or cathodic protection. Information is shown in Table 9. Figures 20 and 21 show probability of damage and life cycle cost in US$ of bad quality structure after beginning repaired by different method considering 0 percent discount rate. Only schedule of repairing that result in the lowest life cycle of repaired case is shown in the Figure 22. As shown, life cycle cost of repaired case is significantly lower than in case of no repairing due to the decrease of failure cost. In case of reinforced concrete structure having durability, many repairs are required along the service life of structure.

Table 7 Information of inspected structure

Bridge No.	Location	Age(years)	Cementitious
1	Samutprakan Province	43	OPC Type 1
2	Chonburi Province	5	OPC Type 5
3	Chantaburi Province	1	OPC Type 1 + Fly ash

Source: Sancharoen *et al*. (2008)

Table 8 Inspection results of surface chloride content and chloride diffusion coefficient

No.	Results	No. of Data			
		1	2	3	4
1	C_o(kg/m^3)	11.17	7.82	7.14	-
	D_c(kg/m^2)	0.82	2	1.19	-
2	C_o(kg/m^3)	10.6	14.1	17.1	19.5
	D_c(kg/m^2)	0.75	0.58	0.39	0.36
3	C_o(kg/m^3)	8.6	5.58	14.39	6.52
	D_c(kg/m^2)	0.35	0.73	0.75	1.01
	C_o(kg/m^3)	3.95	11.13	-	-
	D_c(kg/m^2)	1.32	1.38	-	-

Source: Sancharoen *et al*. (2008)

Table 9 Conclusion of performance and cost of repairing option

No.	Method	Performance		Fixed cost, US$	Variable cost, US$/m^2	Annual cost, US$/m^2
		Mean, cm^2/year	COV			
1	Patching	D_p = 0.175	20%	1500	200	-
2	Patching	D_p = 0.175	20%	1500	200	-
	Surface coating	C_s = 0.5 mm D_s = 1.0E-3 cm^2/year	10%	500	200	-
3	Patchhing	D_p = 0.175	20%	1500	200	-
	Cathodic protection	Titnium mesh (service life 30 year)		6500	150	2.5

Source: Sancharoen *et al*. (2008)

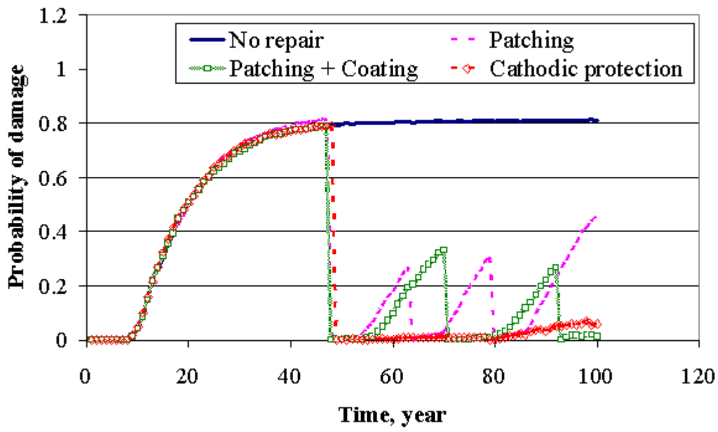


Figure 20 Probability of damage of bad quality structure with different repairing method

Source: Sancharoen *et al*. (2008)

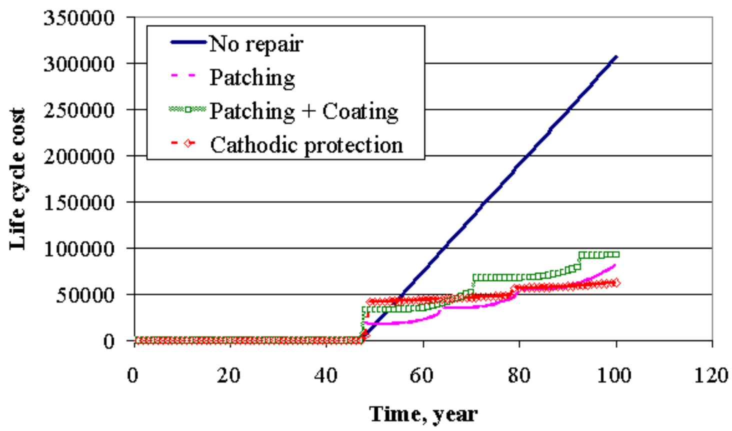


Figure 21 Life cycle cost of bad quality structure with different repairing method

Source: Sancharoen *et al*. (2008)

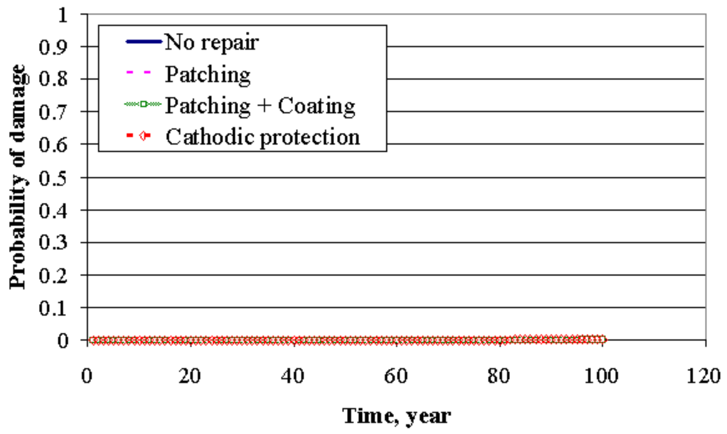


Figure 22 Probability of damage of good quality structure with different repairing method

Source: Sancharoen *et al*. (2008)

MATERIALS AND METHOD

Optimization of bridge management and maintenance planning begin a success or a failure is much more complex. However, the concept of the optimization of bridge can be evaluated through performance measures that can be developed from research literature where various success criteria can be identified.

Materials

The minimum requirements suggested are as follows:

1. Drill pierces the concrete
2. Brush dust
3. The meter
4. A PC which requires
   4.1. CPU: Pentium 90 or higher processors, with minimum 256 MB free RAM (26MB strongly recommended.)
   4.2. Other accessories: Graphics card and monitor with 800 600 resolution(1024 768 recommended), 256 color displays (16 bit high color recommended).
5. Software
   5.1. Operating system: Windows 95 or higher
   5.2. Microsoft office 97 or higher
   5.3. MATLAB software

Methods

The basic element of the methodology consists of five major parts: (1) literature review, (2) developed model (3) bridge testing program, (4) data analysis, and (5) life- cycle cost optimization.

Step 1. Literature review

1. Study on the papers and books to find out bridge deterioration, life-cycle cost, chloride-induce corrosion.
2. Study on the statistics theories to develop model.
3. Study on the MATLAB programming.

Step 2. Developed model

The repaired structure, a hybrid of new old materials, is not expected to be as the new structure. In addition, other factors will affect the reliability (such as its internal degradation, accidental collisions, and aging). In this study, these factors all grouped were ignored when considering deterioration process caused by corrosion of reinforcement which will lead to a reduction in the bar diameter of the reinforcing steel.

The effect of corrosion must be considered in determining the resistance capacity of the structure after a repair as shown Figure 23. Assume that a repair action is undertaken at time T_{pi}. An original structural reliability index of a reinforced concrete structure under age deterioration is $\beta_{r,o}$ and remaining structural reliability index under corrosion damage is $\beta_{r,corr}(T_{pi})$.Assuming the damage was detected and a repair was made, the corresponding structural reliability index is denoted as $\beta_{r,rep}(T_{pi})$.

The deterioration rate (α), expressed in terms of the reliability index (β) per year due to the loss of section, can be obtained from the best fit of the deterioration model by using linear equation.

$$\beta_{r,corr}(T_{pi}) = \beta_{r,o} - \alpha \ (T_{pi} - T_i) \tag{35}$$

$$\beta_{r,rep}(T_{pi}) = \beta_{r,corr} + \gamma_i \tag{36}$$

Where $\beta_{r,o}$ = original structural reliability index

$\beta_{r,corr}$ = remaining mean capacity under corrosion damage

$\beta_{r,rep}$ = remaining structural reliability after repair

α = deterioration rate of structure

T_{pi} = time to repair at i

γ_i = a repair action under taken at time T_{pi}

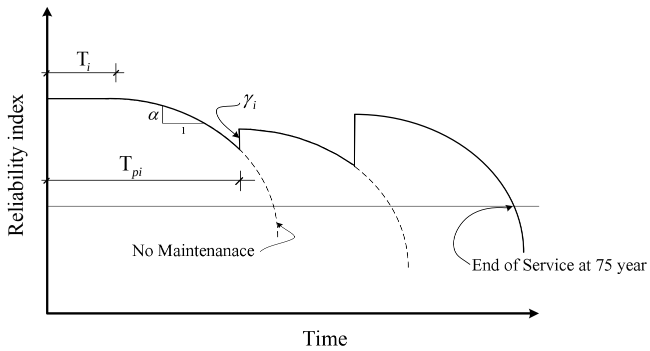


Figure 23 Condition reliability profiles without or with maintenance

Figure 24 shows process of life-cycle cost model that predicts the time to repair of bridge subjected to chloride-induced corrosion based on the structural reliability concept, a stochastic model for evaluating the level of safety and service life of bridge superstructures and substructure. It also highlights the use of the developed model and computer program. It consists of 5 parts: (1) Inspection data and literature of analysis, (2) Evaluation of performance, (3) Deterioration prediction , (4) Cost and effect of maintenance and (5) Optimization of rehabilitation strategy.

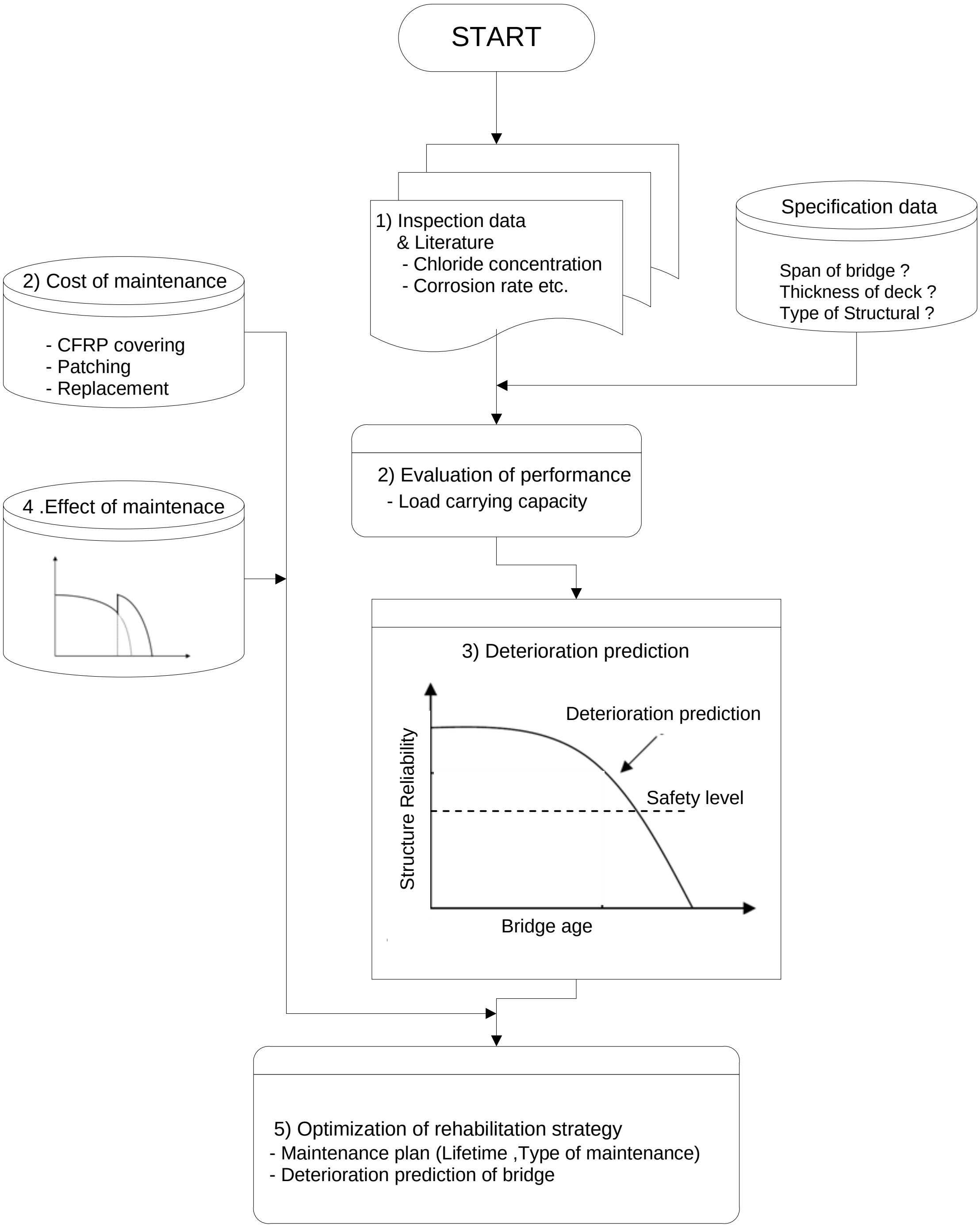


Figure 24 Flowchart showing procedure of life-cycle cost model

Step 3. Bridge testing program

3.1 Selection of bridge structure

The reliability analysis for deterioration prediction was conducted on an example of reinforced concrete bridges located in the southern part of Thailand. The structure shows signs of excessive deterioration due to chloride-induced corrosion, especially in pier areas, after 14 years in service. The structure has a typical span length of 10 m, as presented in Figures 25 and 26. During the field investigation for damage assessment, dimensions of the bridge structure were measured for compliant checks with the structural drawings as presented in Figures 27 and 28.

Figure 25 Example of concrete bridge to investigate

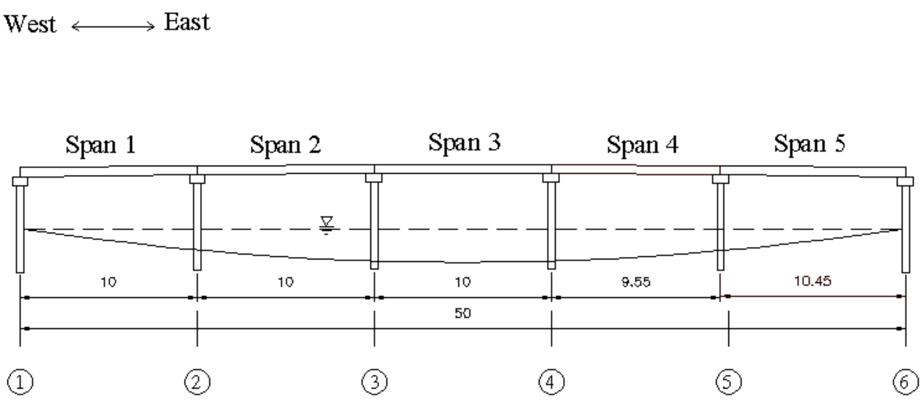


Figure 26 Dimension of bridge example unit meter

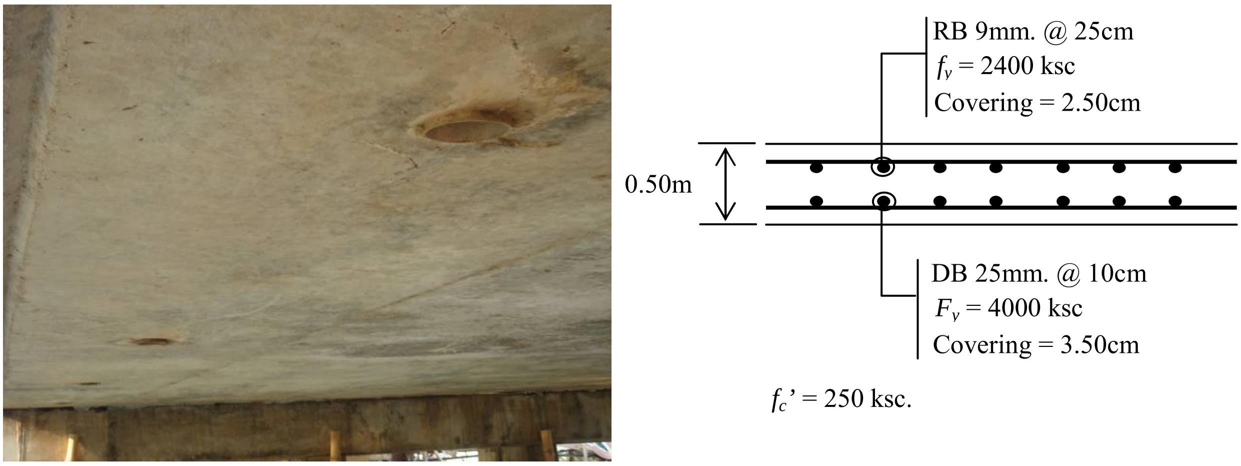


Figure 27 Typical section of concrete bridge deck span length of 10 m

Figure 28 Typical section of column for concrete bridge span length of 10 m

3.2 Powdered Samples for Chloride Content Analysis

The powdered samples extracted from the concrete drilling were analyzed for the acid soluble chloride concentration according to ASTM C1152-111. Powdered samples to be used for chloride content analysis were extracted from five locations on each depth of the samples from bridge as shown in Figure 29. First sample was extracted at surface following with the level 1, 2, 3, and 4 centimeter, respectively. Powder samples removed from the upper 1 to 3 mm of the surface concrete have surface chloride content. For each of the levels picked, powder quantity carries the precedence about 10 gram, after doing introvert type powder for all five examples of concrete as shown in Figures 30 to 32. The location of the drill sample at pier 4 point and one bridge deck is as shown in Figure 29.

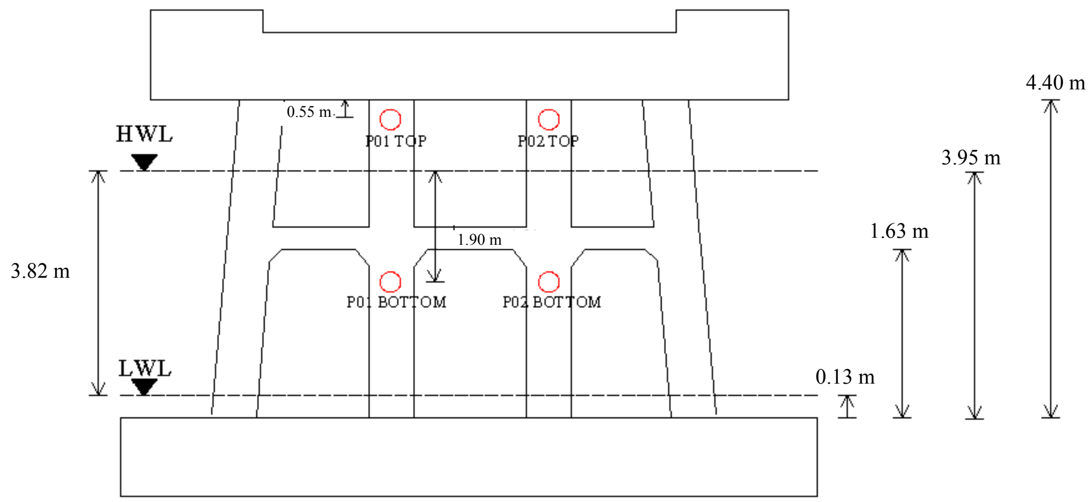


Figure 29 Location to drill for powder samples at pier.

Figure 30 Clean surface and marking of location to drill

Figure 31 Drill concrete at depth level that is wanted then save the powder example

Figure 32 Clean the hole and drill before drilling next level

Step 4. Data analysis

4.1 Time to corrosion initiation

An apparent diffusion process based on Fick's second law is widely used to estimate the time for chloride ions to reach a critical value, initiating corrosion of reinforcing steel. The chloride concentration can be expressed as:

$$C_{(x,t)} = C_O(1 - erf\,\frac{x}{2\sqrt{D_C t}}) \tag{37}$$

Where	$C_{(x,t)}$	=	chloride concentration at depth and time, kg/m3
	C_O	=	surface chloride concentration, kg/m3
	D_C	=	apparent diffusion coefficient, mm2/year
	t	=	time for diffusion, year
	x	=	concrete cover depth, cm
	erf	=	statistical error function.

However, for a given bridge, the values of $C_{(x,t)}$, C_O, D_C, and x are random variables, each of them has their own statistical distributions, means, and variances. A solution of Equation (37) for the diffusion time should include the probabilistic nature of the input variables. The time for corrosion damage to the end of functional service life is likely a random variable as well and depends on the corrosion rate, concrete cover depth, bar spacing, and steel diameter.

Monte Carlo simulation was employed in the study to determine the time to corrosion initiation. The simulation is a general class of repeated sampling methods where a desired response is determined by repeatedly solving a mathematical model using values randomly sampled from the assumed probability distributions of the input variables as shown in Figure 33.

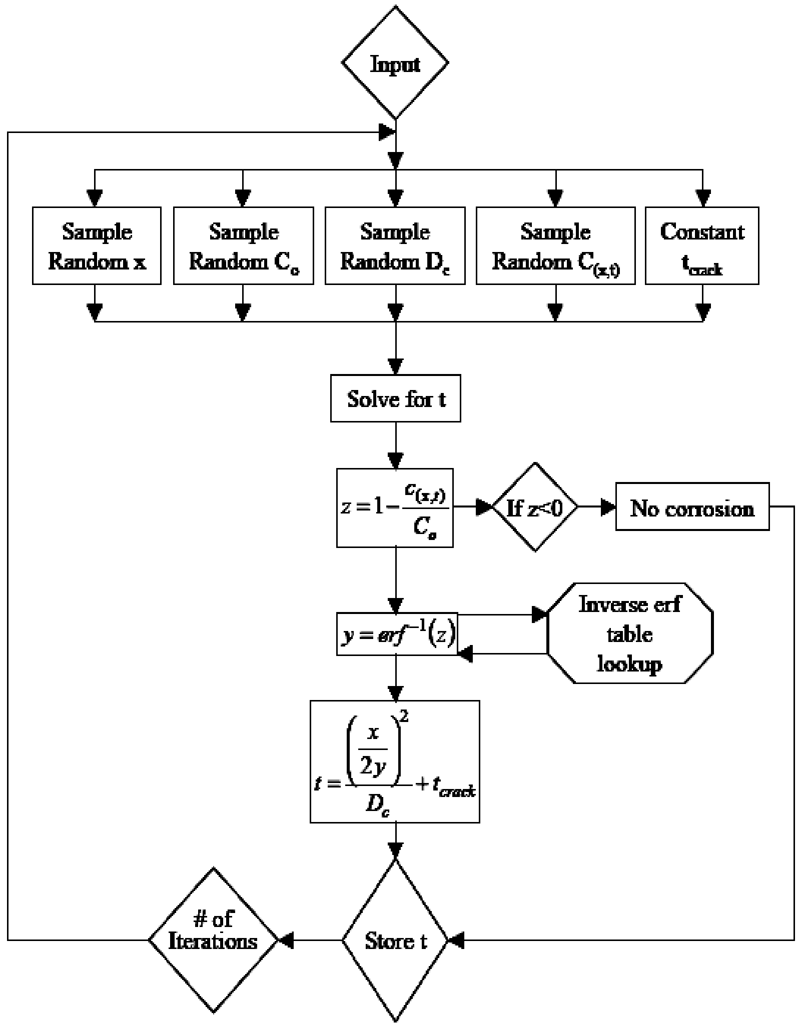


Figure 33 Flow chart to determine time for corrosion
Source: Kirkpatrick (2002)

4.2 Chloride Corrosion Initiation Concentration

The concentration of chloride being necessary to initiate corrosion is not a fixed value, and significant variation exists between structures and within a structure. The chloride concentration, which is necessary to initiate corrosion in reinforced concrete, may be influenced by many factors. Based on literature, a range of 0.6 to 1.2 kg/m^3 (1.0 to 2.0 lb/cy) has been suggested as a conservative estimate for using in the service life model. The uniform distribution with a mean value of 0.9

kg/m^3 and a coefficient of variation of 19% were used for a critical chloride concentration at initiation used in this study (Kirkpatrick, 2002).

4.3 Corrosion damage

After the initiation phase, corrosion will occur and result in a section loss of reinforcing steel bars. A reduction of bond is however not considered herein, as it appears generally to have a negligible effect on bridge reliability. During the propagation phase, a rebar diameter at any given time could be expressed as:

$$D(t) = \begin{cases} D_i & t \le T_i \\ D_i - 2\lambda(t - T_i) & T_i < t \le T_i + (D_i / 2\lambda) \\ 0 & t > T_i + (D_i / 2\lambda) \end{cases} \quad (38)$$

Where $D(t)$ = diameter of the reinforcing steel at time, mm

D_i = initial bar diameter, mm

T_i = time to diffusion, year

λ = corrosion rate at a surface (mm/year)

In this study, to demonstrate the effect of corrosion rate based on literature, five mean corrosions rates are considered. The five corrosion rates used in this study indicating a mean value were 0.045cm/year, 0.064cm/year and 0.083 cm/year for slab and 0.11cm/year and 0.15cm/year for pier with a standard deviation 0.020cm/year, as suggested by Frangopol *et al*. (1997).

From the result the deterioration rate (α) was found to have average of 0.077 per year, 0.104 per year and 0.1275per year for corrosion rate in slab of 0.045 cm/year, 0.065 cm/year and 0.085 cm/year, respectively and 0.356 per year and 0.475 per year for corrosion rate in the pier of 0.045 cm/year and 0.085cm/year, respectively.

Step 5. Life- Cycle cost Optimization

In this study, the Fault Tree Decision was applied for a model in order to represent all possible events associated with no maintenance, minor repair, major repair or replacement action.

The expected total cost of the maintenance is difficult to predict. Traditionally, an engineering cost associated with the maintenance expenditure is used for estimating budgets and planning. The maintenance work is proportional to the size and the age of the bridge. It may become more attractive at some points to replace a bridge rather than spend a large amount of money to maintain it. Maintenance cost increasing with time is considered as constant value is the study. An estimate of the cost must consider the effect of time. For a given bridge, the cost of maintenance at any time t , C_{REP} may be assumed a linear function defined as:

$$C_{REP} = \sum_{i=1}^{n} \frac{C_{R_i}}{(1+r)^{t_{R_i}}} \tag{39}$$

Where	C_{REP}	=	cost of all repairs, US$
	r	=	discount rate of money
	t_{R_i}	=	time of repair, year

Assuming a service life of 75 years for this study, discount rate of money is difficult to predict since it depends on the economical conditions during the lifetime of the structure. The minimum of the total cost to repair can be expressed as:

$$\min C_T = C_{REP} \tag{40}$$

Where	$\min C_T$	=	minimum cost of all repairs, US$

RESULTS AND DISCUSSION

This chapter provides the methodological descriptions for the development of project prioritization framework. It also highlights the use of the developed computer program. It consists of 4 parts: (1) chloride content analysis, (2) finding time to corrosion initiation, (3) finding deterioration of concrete bridge structure, (4) life-cycle cost optimization, as presented in Figure 34.

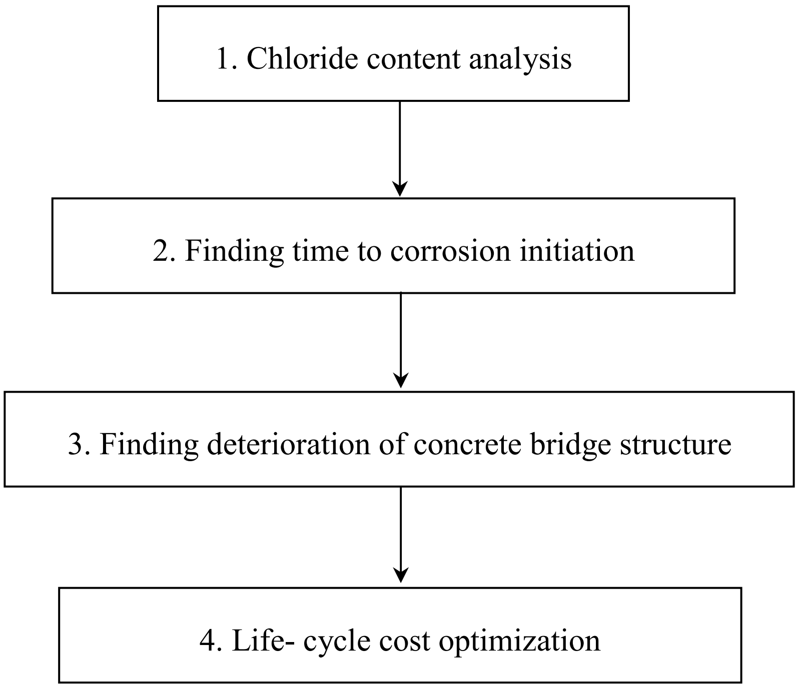


Figure 34 Flowchart showing procedure used to calculate life cycle cost in the study

1. Chloride content analysis

Chloride concentrations determined for all concrete powder samples were collected from five depths for three locations of the concrete bridge structure. Chloride concentrations from slab and pier versus depth are presented in Figures 35 and 36 and Tables 10 to 12 respectively. Powdered samples to be used for chloride content analysis were extracted from three locations on each depth of the samples from bridge as shown in Figure 29. The concrete powder samples for chloride concentration determination at 14 years were collected from the concrete bridge structures that were placed in the chloride environment zones. Once the chloride ion concentrations were measured, diffusion coefficients were calculated for all concrete powder samples. Regression analysis of the chloride profiles obtained field data was conducted using Fick's second law of diffusion, given by Equation 1.

Table 10 Test result of acid soluble chloride concentration of concrete for slab

Sample	Depth(cm)	Time(year)	Wt. of cement	$AgNo_3$	%Cl	Wt. Cl/m^3
Slab	0	14	1.6871	0.920	0.097	2.328
Slab	1	14	1.5506	0.240	0.028	0.672
Slab	2	14	1.3240	0.110	0.015	0.360
Slab	3	14	1.2346	0.020	0.003	0.072
Slab	4	14	1.6488	0.100	0.010	0.240

Table 11 Test result of acid soluble chloride concentration of concrete for pier (splash zone) sample 1.

Sample	Depth(cm)	Time(year)	Wt. of cement	$AgNo_3$	%Cl	Wt. Cl/m^3
P01	0	14	1.4743	1.000	0.120	2.880
P01	1	14	1.3024	0.290	0.039	0.936
P01	2	14	1.4290	0.170	0.021	0.504
P01	3	14	1.3595	0.200	0.027	0.648
P01	4	14	1.6218	0.180	0.020	0.480

Table 12 Test result of acid soluble chloride concentration of concrete for pier (splash zone) sample 2

Sample	Depth(cm)	Time(year)	Wt. of cement	$AgNo_3$	%Cl	wt. Cl/m^3
P02	0	14	1.2304	0.690	0.099	2.376
P02	1	14	1.4310	0.040	0.063	1.512
P02	2	14	2.2580	0.250	0.020	0.480
P02	3	14	1.0873	0.080	0.013	0.312
P02	4	14	1.0630	0.080	0.013	0.312

Since the chloride concentrations were measured at four depths, the “Least Square” was employed to obtain the best fit of the diffusion coefficient. Calculated diffusion coefficients corresponding to the minimum of the sum of squared errors, are presented in Table 13. According to the variance of chloride concentration value at various weather atmospheres, the chloride concentrations at surface were neglected during the regression analysis. The chloride content of powdered concrete samples were also analyzed for the acid soluble chloride concentration according to ASTM C1152-111. The “ near surface” chloride concentration (C_o) was found to have an average of 0.993 kg/m^3 for slab and 23.309 kg/m^3 for concrete slab. The apparent diffusion coefficient (D_c) also had an average of 18.164 mm^2/year for slab and 100.842 mm^2/year for pier.

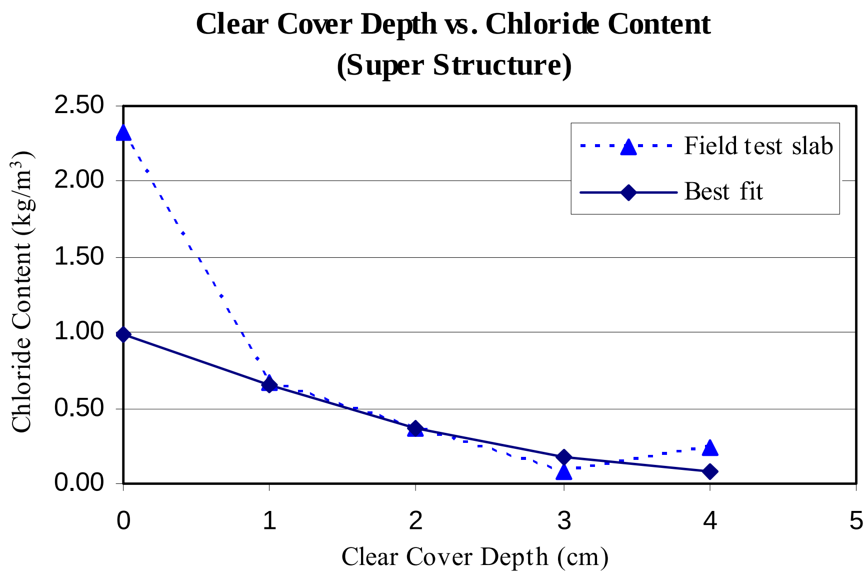


Figure 35 Chloride concentration of concrete bridge from slab

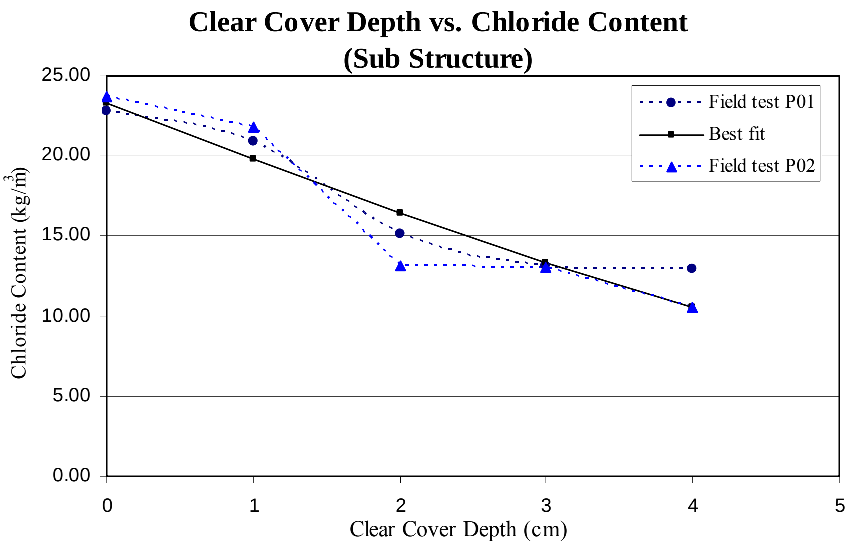


Figure 36 Chloride concentration of concrete bridge from pier in splash zone

2. Finding time to corrosion initiation

Time to corrosion initiation of corrosion in concrete is modeled using Fick's law of diffusion. Monte Carlo simulation was employed in the study to determine the time to corrosion initiation of reinforcing steel in bridge components. The simulation is a general class of repeated sampling methods where a desired response is determined by repeatedly solving a mathematical model using values randomly sampled from the assumed probability distributions of the input variables in Table 13.

Figures 37 and 39 shows the probability density function (PDF) for the time when corrosion was expected to start on surface of reinforcing steel in slab and pier. Corrosion initiation time is a random variable, which in turn is a function of other independent random variables. Independent random variables listed in the Table 13, x, D_c, C_o, and C_{cr}, are the bar clearance for reinforcing steel, diffusion coefficient, surface chloride concentration, and critical chloride concentration level on steel surface for the onset of corrosion, respectively. Based on literature, the uniform distribution with a mean value of 0.9 kg/m3 and a coefficient of variation of 19% was used for a critical chloride concentration at initiation.

Table 13 Summary of statistical parameters from field assessment and literature

Parameter	Mean(x)	COV.	Distribution
$C_{(x,t)}$	0.9kg/m^3	0.19 [1/]	Uniform (0.6-1.2) [1/]
C_O	0.993 kg/m^3	0.197 [1/]	Gamma (slab) [1/]
	23.309 kg/m^3	0.197 [1/]	Gamma(pier splash zone) [1/]
D_C	18.164 mm^2/year	0.443 [1/]	Gamma (slab) [1/]
	100.842 mm^2/year	0.433 [1/]	Gamma (pier splash zone) [1/]
x	26mm	0.199 [2/]	Normal (slab) [2/]
	70mm	0.182 [2/]	Normal (pier splash zone) [2/]

Source: Kirkpatrick (2002) [(1)]; Amatayakul (1988) [(2)]

The probability density functions (PDF) of the initiation times provided by 218 simulations. Due to a probabilistic nature of the problem, a certain confidence level must be selected to obtain an estimate of the time to initiation. By using a safety level of 97.5% or a 2.5 percent probability of failure, the estimated times to corrosion of the bridge structure were found to be 28 years for reinforced concrete slab and 2 years for pier as shown in Figures 38 and 40, respectively, which are in the range of Kirkpatrick (2002) study for slab.

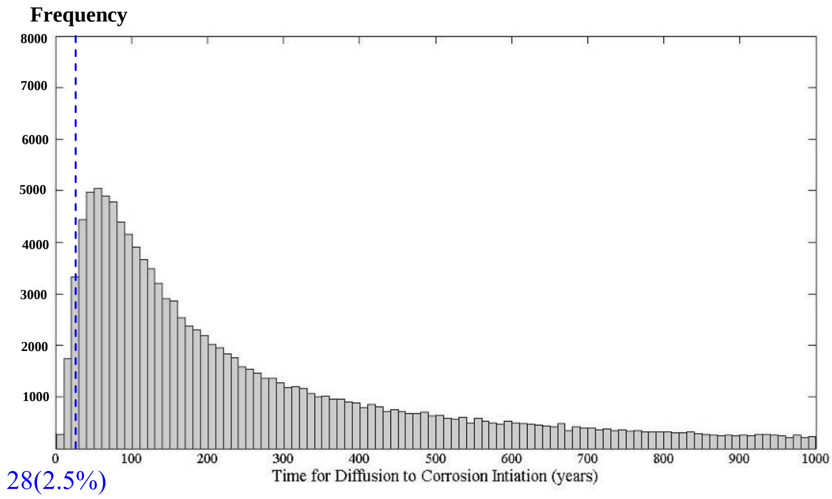


Figure 37 Probability distribution of time for diffusion to corrosion initiation, $C_{(x,t)}$ (slab)

The result shows in Figure show present high dispersion because constant coefficient of variation (COV) of D_c, C_o were assumed during the analysis. As a result, small standard deviation of these two parameters for slab were obtained. Therefore, high dispersion of time for diffusion to corrosion initiation was used to compensate for a high value of standard deviation of threshold chloride concentration ($C_{(x,t)}$) in the Fick's law (Equation (2)).

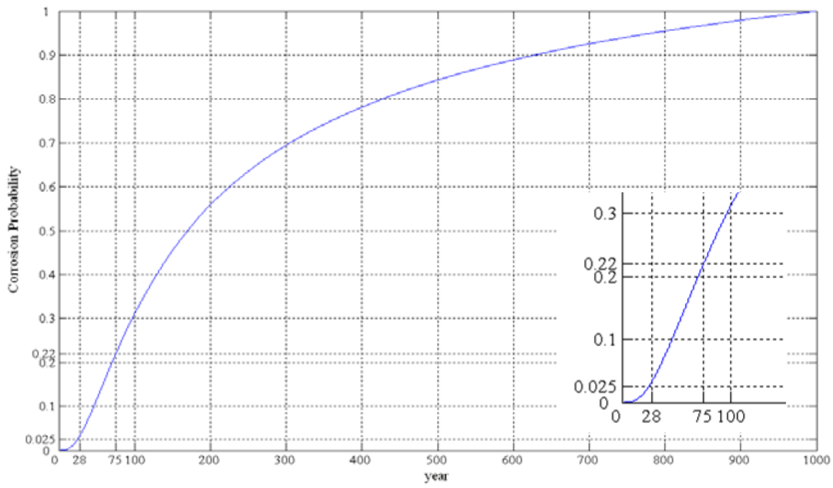


Figure 38 Cumulative-time failure probability for diffusion to corrosion initiation, $C_{(x,t)}$ (slab)

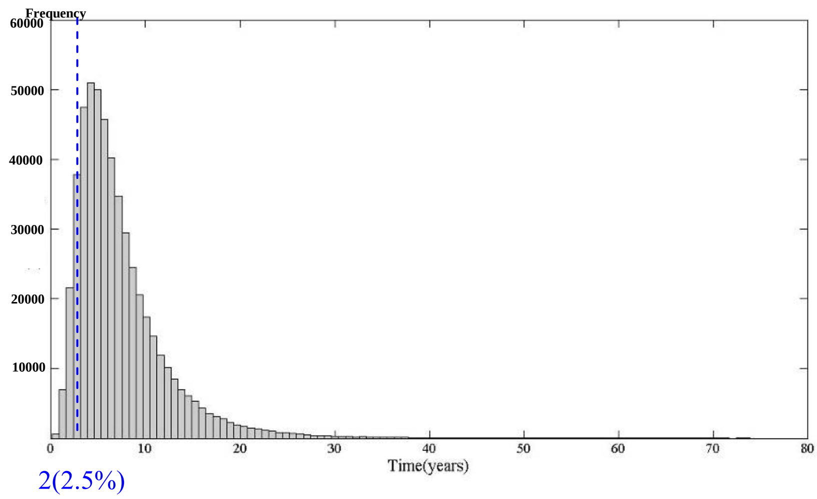


Figure 39 Probability distribution of time for diffusion to corrosion initiation, $C_{(x,t)}$ (pier)

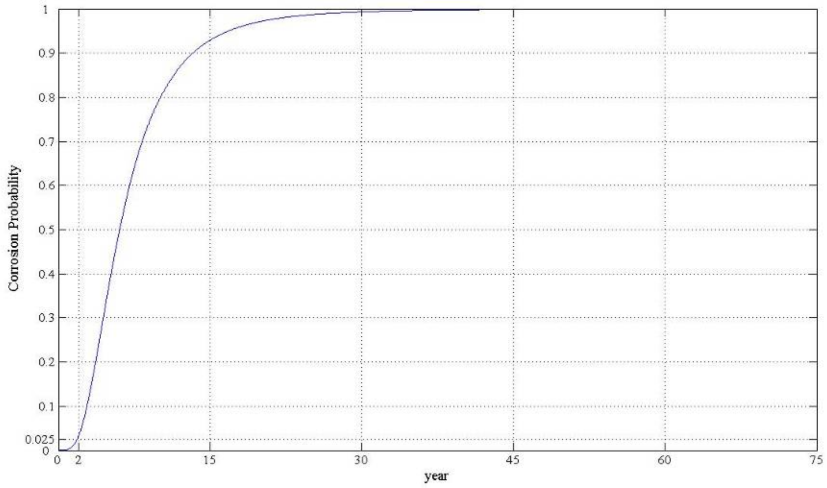


Figure 40 Cumulative-time failure probability for diffusion to corrosion initiation, $C_{(x,t)}$ (pier)

3. Finding deterioration of concrete bridge structure

The deterioration due to chloride-induced corrosion of the steel reinforcement was identified as the dominant influence on the strength of the concrete structure. The inherent uncertainties in the material properties and environmental effects were incorporated in the model to determine the reduction in capacity due to a loss in cross-sectional areas of reinforcing steel. This study considers deterioration due to chloride induced corrosion of the steel reinforcement concrete slab and pier bridges under marine environment in Thailand. Based on the structural reliability concept, a service life of bridge structures can be obtained with a certain level of safety. The corrosion rate of the structure members was predicted using the deterioration obtained from the literature (Frangopol, 1997; Enright *et al.*, 1998).

Structural reliability analysis can be employed for evaluating the probability of failure of structural elements for a given limit state at any time during their service life. It can incorporate inherent uncertainties in random variables in the calculation. A limit state function was utilized to represent the performance of structural elements in terms of a number of basic random variables. The statistical parameter mean values and coefficients of variation used for the bridge structure are presented in Table 14. The levels of safety corresponding to the 1.3×HS20-44 live load model provided in the AASHTO Specifications (1996) were determined.

Load models are an important part of the code. The major components of bridge loads are dead load, live load, dynamic load (impact). Therefore, these three load components were considered in this study. Nominal (design) values of load components were calculated according to AASHTO Standard (1996) as shown in Figures 41 and 42.

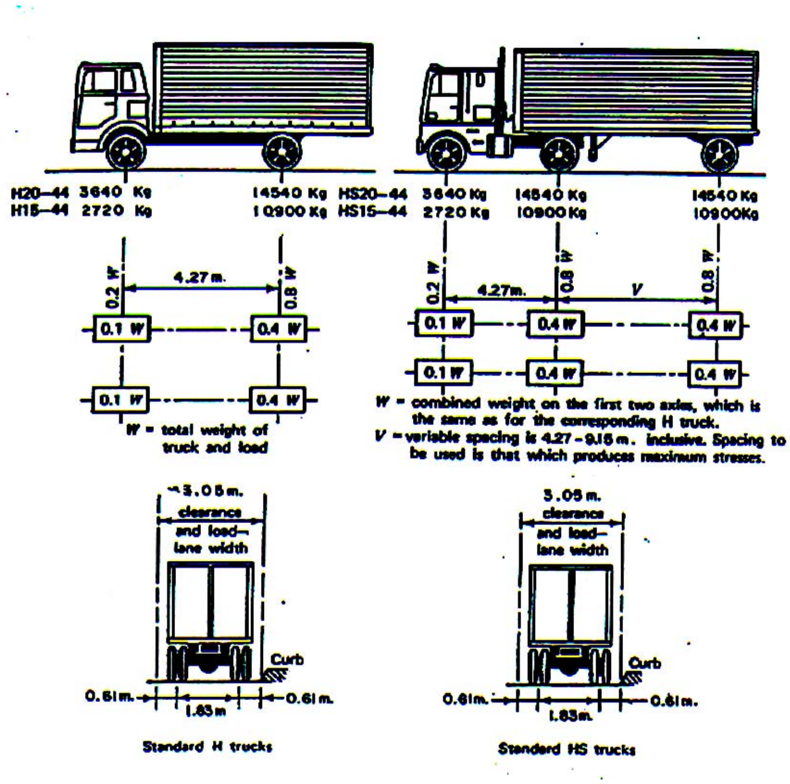


Figure 41 AASHTO Standard truck loading

Source: AASHTO specifications (1996)

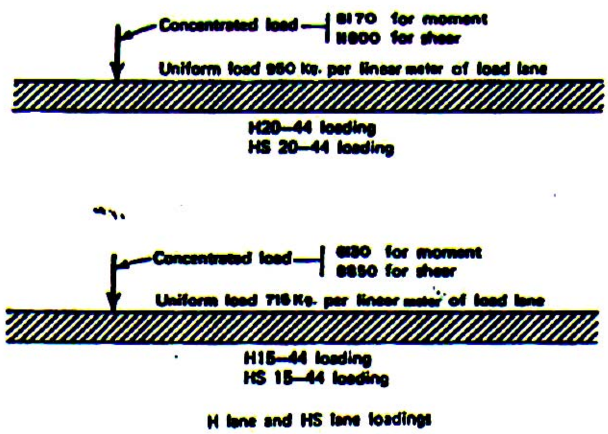


Figure 42 AASHTO Standard lane loading

Source: AASHTO specification(1996)

Table 14 Random variable used in reliability analysis of concrete bridge

Definition	**Notation**	**Mean**	**COV**	**Distribut.**
Dead load				
Bias factor of cast-in-place concrete	λ_1	1.05[1/]	0.10[1/]	normal[1/]
Bias factor of asphalt surface	λ_2	1.10[1/]	0.25[1/]	normal[1/]
Bias factor of miscellaneous	λ_3	1.05[1/]	0.10[1/]	normal[1/]
Live load				
Bias factor of live load (slab)	λ_{LL}	1.766[1/]	0.12[1/]	normal[1/]
Bias factor of live load (column)	λ_{LL}	1.55[1/]	0.12[1/]	normal[1/]
Impact factor (dynamic factor)	λ_{IM}	0.17[1/]	0.80[1/]	normal[1/]
Resistance				
Compressive strength of concrete, ksc	f_c'	250[2/]	0.166[2/]	normal[2/]
Yield strength of steel (SD40), ksc	Fy	4614[2/]	0.042[2/]	Lognor[2/]
Yield strength of steel (SD24), ksc	F_y	3467[2/]	0.068[2/]	Lognor[2/]
Diameter of compression steel, mm	ϕ_t	24.94[2/]	0.006[2/]	Lognor[2/]
Diameter of tension steel, mm	ϕ_c	9.09[2/]	0.0099[2/]	lognor[2/]
Covering of compression steel, m	dc	0.022[2/]	0.197[2/]	normal[2/]
Covering of tension steel, m	dt	0.026[2/]	0.019[2/]	normal[2/]
Covering of pier, m	x	0.07[2/]	0.182[2/]	normal[2/]
Dimensions (thk., wide, depth)			0.016[2/]	normal[2/]

Source: Nowak (1995)[(1)]; Amatayakul (1988)[(2)]

It was assumed during the analysis that the end of service life will occur if a reliability index (β) falls below a target reliability value of 2.0. By utilizing the Advance First Order Second Moment Method (AFOSM), the levels of safety or reliability indexes for bridges' slab and pier at a given year in service were determined. The flexural and compression capacities were used in the calculation to determine the level of safety for the slab and pier, respectively. The deterioration rate (α) was found to have average of 0.077 per year, 0.104 per year and 0.1275per year with corrosion rate of 0.045 cm/year, 0.065 cm/year and 0.085 cm/year, respectively for slab and 0.356 per year and 0.475 per year with corrosion rate of 0.045 cm/year and 0.085cm/year, respectively for pier.

4. Life- Cycle Cost Optimization

Several methods have been used to protect the reinforcing steel from chloride corrosion attack in concrete bridge decks. The methods include low permeable concrete to slow the ingress of chlorides, polymer overlays and deck, increased concrete cover depth, cathodic protection, and alternative strengthening reinforced concrete structure. Repair and rehabilitation methods are several important factors which influence the decision in any repair and rehabilitation project. Some of these factors are :

1) The nature, extent, and severity of the defect
2) The effect of the proposed repair method on the service life of the bridge
3) The extent to which the repair process will disrupt traffic flow
4) The availability of funds

Minor repair, major repair, and replacement are the main repair procedures commonly used for the three defects under consideration. These procedures can be categorized on the basis of their extent and depth of deterioration as follows:

1) Minor repair is low budget repair, which is recommended when the rebar are exposed and 5% of damage section loss. In this method, the deteriorated concrete is saw cut and is removed either by pneumatic hammer or by hydro demolition, then the surface is cleaned and the repair material is applied and cured.

2) Major repair is required when the deterioration has gone deeper than the steel reinforcement. The deteriorated material is chipped off and at least 1 in. of the concrete under the reinforcing steel is removed and 25% of damage section loss. The exposed bars are then thoroughly cleaned by sand blasting or hydro demolition. The reinforcing bars are also checked for any defects and are subsequently repaired. The

repair material is poured once the bars are cleaned and strengthening. In this study, the carbon fiber reinforcement plate CFRP recover attaching method was considered.

3) Replacement is a treatment option with the highest initial cost and this should always be treated as a last alternative. Both protective and non- protective treatments should be used to delay the deck replacement. deck replacement maybe the better option when the deterioration of the concrete is deeper and greater than half the depth of the slab.

Different maintenance types are considered in this study for enhancing performance of the deteriorating bridge deck slabs patching injection (minor maintenance), carbon fiber reinforcement plate CFRP recover attaching (major maintenance), and total replacement. Rigorously, effects of maintenance interventions on reliability profiles require detailed reliability analysis of bridge deck slabs upon application of different maintenance interventions. In this study, in order to perform maintenance prioritization with reasonable computational expenses, maintenance effects as well as the associated unit costs are provided in Table 15, respectively.

Patching injection is the least costly maintenance type among the other alternatives. It injects epoxy resin into voids and seals cracks in concrete, which repairs the aging deck slabs by reducing the corrosion of reinforcement due to exposure to the open air. In this study, maintenance types instantly improve the bridge reliability level by various amounts upon application. Carbon fiber-reinforced polymer (CFRP) composites are increasingly being used in civil infrastructure applications. Their application in composite bridge decks offers several benefits over conventional materials including lightweight, fast installation, high durability, high corrosion and fatigue resistance, and higher live load capacity.

Normally, a unit cost of typical bridge construction in Thailand is approximately equal to 600US$/m^2. Then, minor repair and major repair of unit cost based on NCHRP are quarter and half of bridge construction cost, respectively.

A repair action (γ_i) in Equation 36 for minor repair and major repair the slab reliability indices by a maximum of 0.7 and 2.0, respectively. The reliability indices value are determined by using 5% and 25% of major and minor damage section loss calculation based on AASHTO manual for condition evaluation of bridge. As a tradeoff, unit costs associated with these two maintenance types are 150US$/m^2 and 300US$/m^2, respectively.

Only one type of maintenance considers analyzed life-cycle cost for pier because of characteristic maintenance pier. A repair action (γ_i) for pier reliability recover the initial reliability level. The reliability indices value is based on replacement new material in maintenance pier. As a tradeoff, unit costs associated with that maintenance types are 100US$/m^2.

Table 15 Different maintenance types

Maintenance type	**Improvement of concrete bridge reliability index**	**Unit cost (US$/m^2)**
Do nothing	0	0
Minor repair (Patching)	0.7	150
Major repair (CFRP recover)	2.0	300
Major repair (for pier only)	Recover the initial reliability level	100

Figures 43 to 50 present an optimization repair strategy that can be developed using all feasible combinations of the options listed in Table 15. In this study, to demonstrate the effect of corrosion rate, three mean corrosions which were 0.045cm/year, 0.064cm/year and 0.083 cm/year with a standard deviation 0.020cm/year, as suggested by Frangopol *et al*. (1997) were considered. The bridge was evaluated every 2 years, a discount rate 6% and required reliability index for bridge greater than 2.0 according AASHTO manual for condition evaluation of bridge.

Figures 43, 44, 47 to 49, and 52 to 54 show reliability profiles with maintenance improved for concrete slab. The optimization of bridge repair strategy is determined as a function of the desired service life extension of the bridge from Tables 16 to 18, where all feasible options and their associated costs are considered. The costs were computed based on cost listed in Table 15. The results show that the maintenance is required when reliability index has deteriorated to nearly reliability, target which is different from the other literature (Frangopol *et al.*, 1997) because procedure to determine maintenance cost due to maintenance action are different.

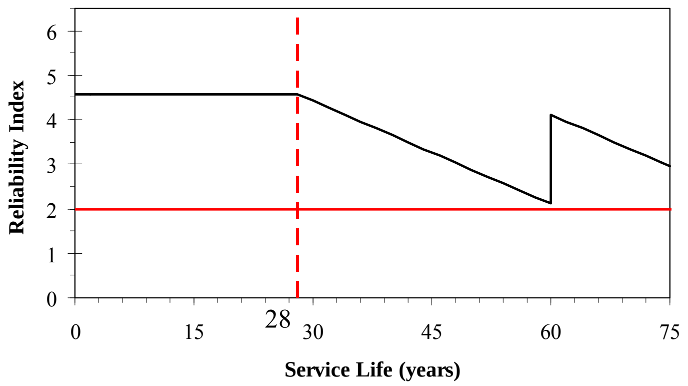


Figure 43 Condition reliability profiles with major maintenance improvement for slab (corrosion rate 0.045mm/year)

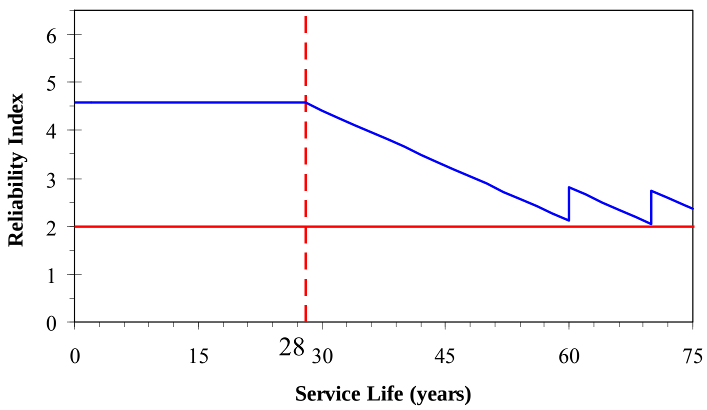


Figure 44 Condition reliability profiles with major maintenance improvement for slab (corrosion rate 0.045mm/year)

Tables 16 to 18 and Figures 45 50 and 55 show, expected total cost, time to repair and number of repair concrete bridge deck with difference maintenance. As indicated, alternative 3 is the most economical because it considered various maintenance consisting of minor and major. Alternative 1 and 2 considered only minor maintenance or major maintenance, respectively. Figures 46, 51, 56 and 59 were shows the expected annual repair cost for bridge deck with various value corrosion rate. It was found that to have average of 23.06 US$/m^2/year and 20.39 US$/m^2/year with alternative 1(minor repair) and 2(major repair) respectively for corrosion rate in slab is 0.045 cm/year 27.92 US$/m^2/year, 26.32 US$/m^2/year and 26.66 US$/m^2/year with alternative 1 (minor repair), 2 (major repair) and 3 (various repair) respectively for corrosion rate in slab is 0.065 cm/year 34.31 US$/m^2/year, 31.06 US$/m^2/year and 30.71 US$/m^2/year with alternative 1 (minor repair), 2 (major repair) and 3 (various repair) respectively for corrosion rate in slab is 0.085 cm/year and 10.76 US$/m^2/year and 13.16 US$/m^2/year for corrosion rate in the pier of 0.011 cm/year and 0.015cm/year, respectively.

Table 16 Expected cost of repair concrete bridge deck with 75 year service life (corrosion rate 0.045mm/year)

No. of Repair	Alternative 1: "Minor repair"		Alternative 2: "Major repair"		Alternative 3: "Various repairs"		
	Year	Unit Cost (US$/m^2)	Year	Unit Cost (US$/m^2)	Year	Unit Cost (US$/m^2)	Description
1	60	4.55	60	9.09	60	4.55	Minor
2	70	2.54			70	2.54	Minor
Total		**7.09**		**9.09**		**7.09**	

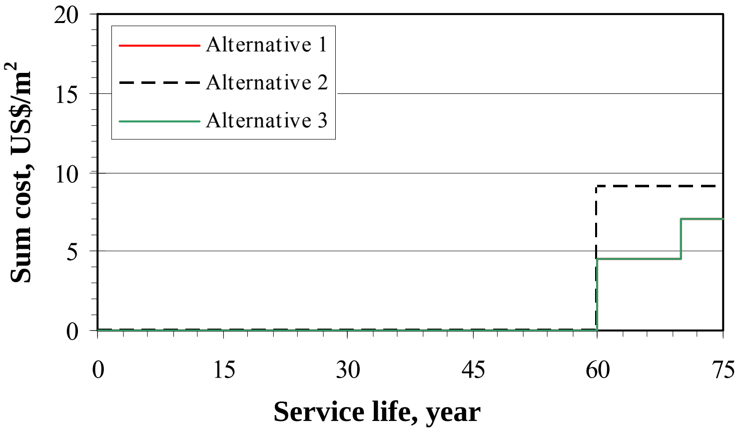
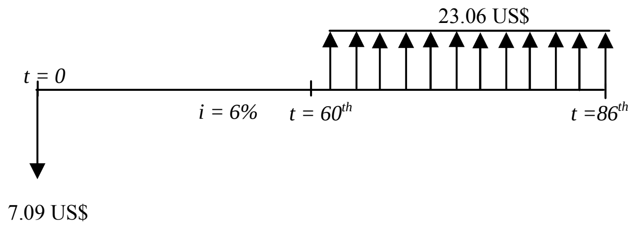
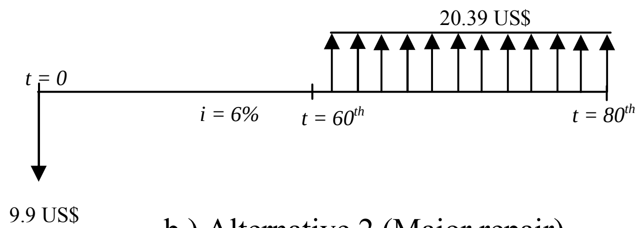


Figure 45 Relationship between sum cost of maintenance and service life for slab (corrosion rate 0.045mm/year)



a.) Alternative 1 (Minor repair)



b.) Alternative 2 (Major repair)

Figure 46 A time line for expected annual cost of repair concrete bridge deck (corrosion rate 0.045mm/year)

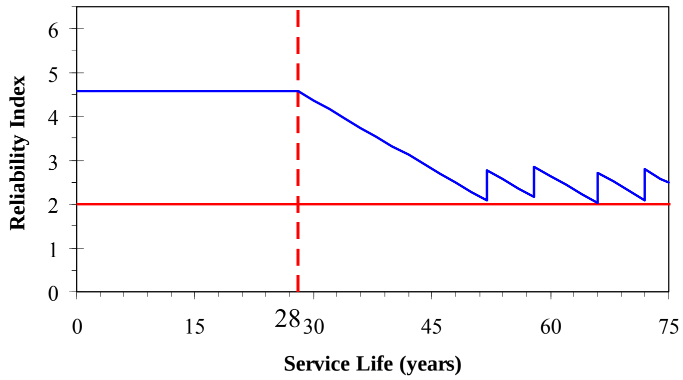


Figure 47 Condition reliability profiles with major maintenance improvement for slab (corrosion rate 0.065mm/year)

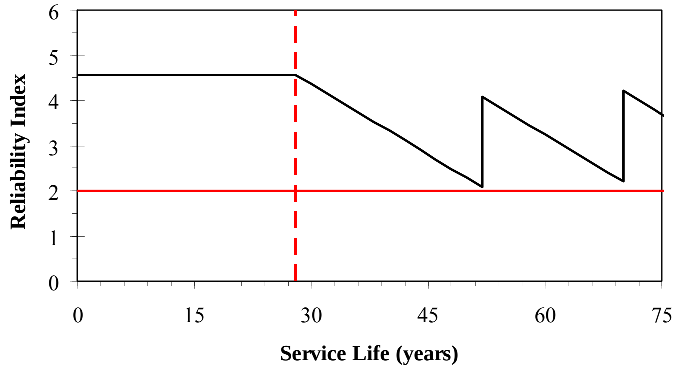


Figure 48 Condition reliability profiles with major maintenance improvement for slab (corrosion rate 0.065mm/year)

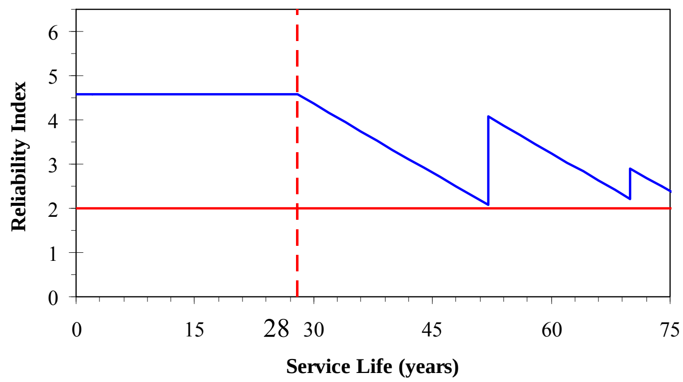


Figure 49 Condition reliability profiles with major maintenance improvement for slab (corrosion rate 0.065mm/year)

Table 17 Expected cost of repair concrete bridge deck with 75 year service life (corrosion rate 0.065mm/year)

No. of Repair	Alternative 1: "Minor repair"		Alternative 2: "Major repair"		Alternative 3: "Various repair"		
	Year	Unit Cost (US$/m^2)	Year	Unit Cost (US$/m^2)	Year	Unit Cost (US$/m^2)	Description
1	52	7.25	52	14.49	52	14.49	Major Repair
2	58	5.11	72	4.52	72	2.26	Minor Repair
3	66	3.21					
4	74	2.26					
Total		**17.82**		**19.01**		**16.75**	

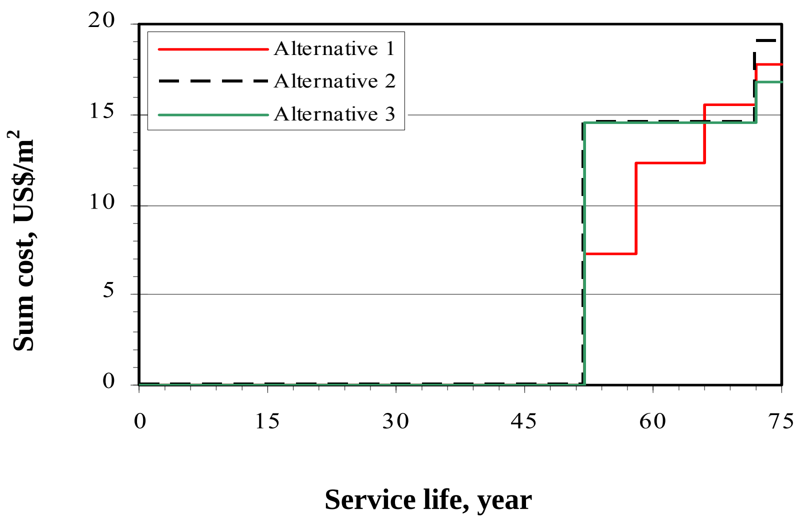


Figure 50 Relationship between sum cost of maintenance and service life for slab (corrosion rate 0.065mm/year)

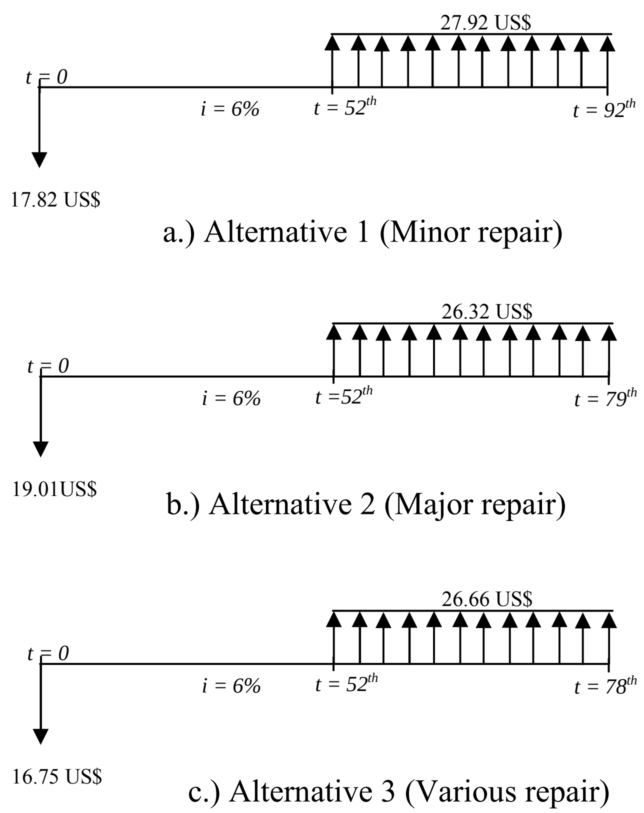


Figure 51 A time line for expected annual cost of repair concrete bridge deck (corrosion rate 0.065mm/year)

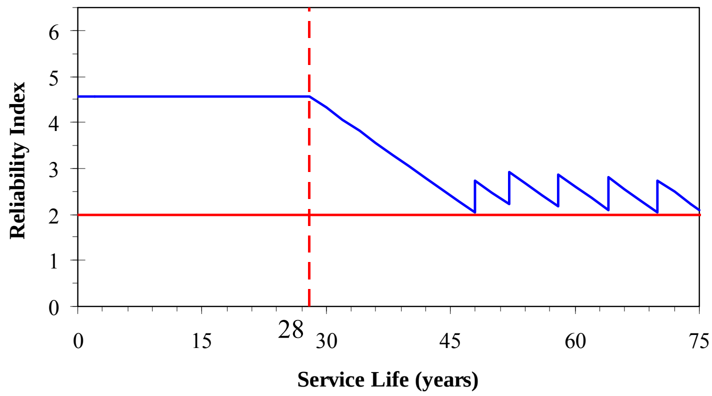


Figure 52 Condition reliability profiles with major maintenance improvement for slab (corrosion rate 0.085mm/year)

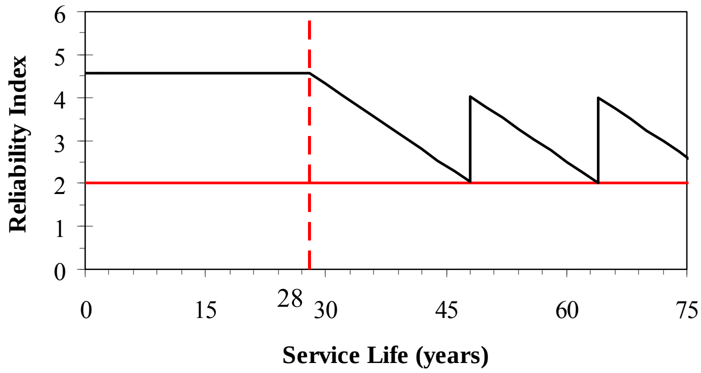


Figure 53 Condition reliability profiles with major maintenance improvement for slab (corrosion rate 0.085mm/year)

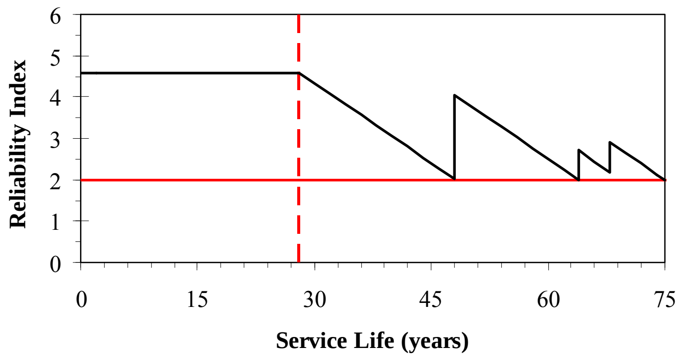


Figure 54 Condition reliability profiles with major maintenance improvement for slab (corrosion rate 0.085mm/year)

Figures 57 and 58 show reliability profiles with maintenance improved for concrete pier. The optimization of bridge repair strategy, which is as a function of the desired service life extension of the bridge from Table 18, where all feasible option and their associated costs are considered. The costs were computed based on cost list in Table 15.

Table 18 Expected cost of repair concrete bridge deck with 75 year service life (corrosion rate 0.085mm/year)

No. of Repair	Alternative 1: "Minor repair"		Alternative 2: "Major repair"		Alternative 3: "Various repair"		
	Year	Unit Cost (US$/m^2)	Year	Unit Cost (US$/m^2)	Year	Unit Cost (US$/m^2)	Description
1	48	9.15	48	18.30	48	18.30	Major Repair
2	52	7.25	62	8.09	64	3.60	Minor Repair
3	58	5.11			68	2.85	Minor Repair
4	64	3.60					
5	70	2.54					
Total		**27.65**		**26.39**		**24.75**	

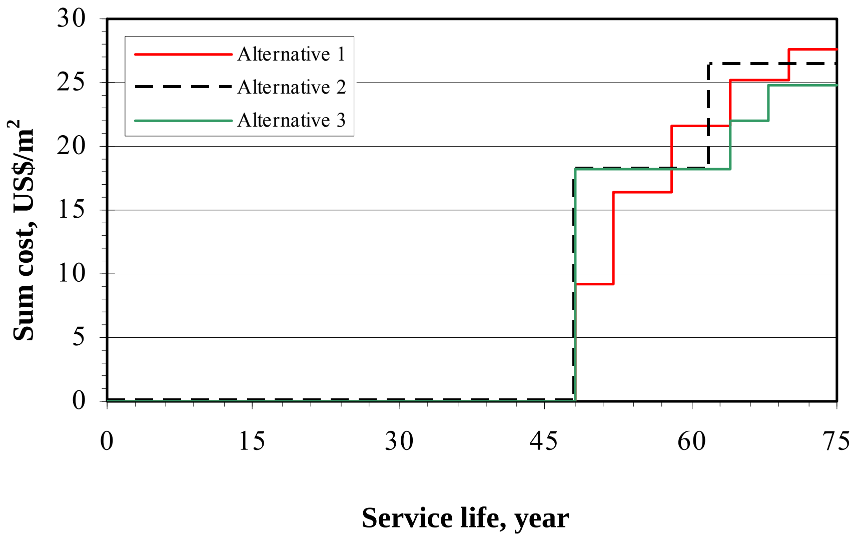


Figure 55 Relationship between sum cost of maintenance and service life for slab (corrosion rate 0.085mm/year)

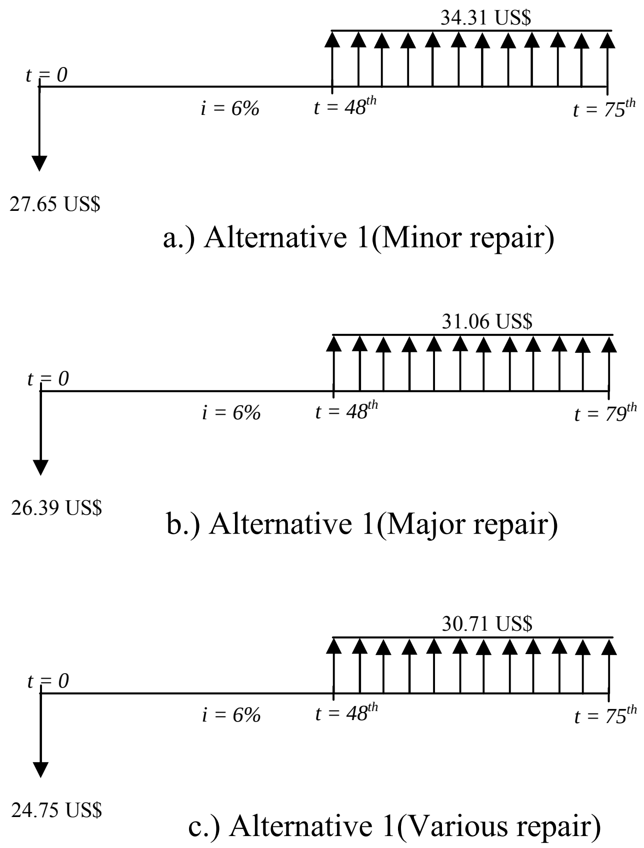


Figure 56 A time line for expected annual cost of repair concrete bridge deck (corrosion rate 0.085mm/year)

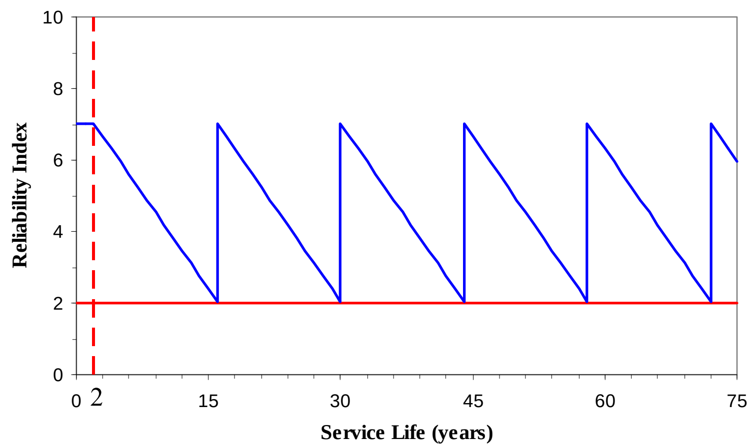


Figure 57 Condition reliability profiles with major maintenance improvement for pier (corrosion rate 0.11mm/year)

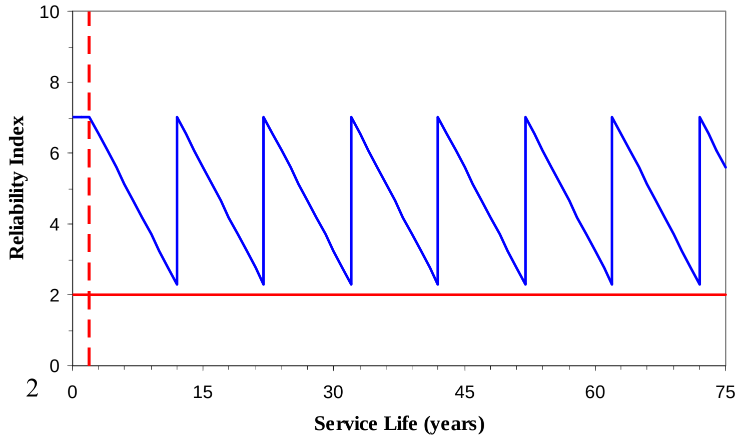
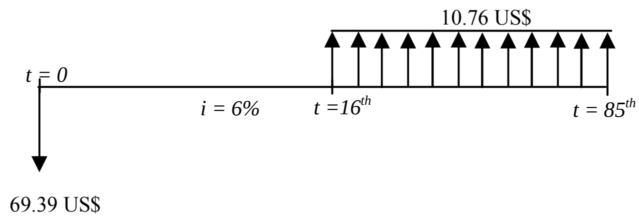


Figure 58 Condition reliability profiles with major maintenance improvement for slab (corrosion rate 0.15mm/year)

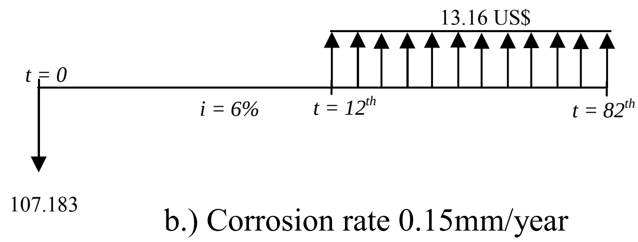
Table 19 shows expected total cost, time to repair and number of repair pier. In this study, to demonstrate the effect of corrosion rate, two mean corrosions which were 0.11cm/year and 0.15 cm/year with a standard deviation 0.020cm/year are considered for pier. The bridge was evaluated every year, a discount rate 6% and required reliability index for bridge grater than 2.0 according AASHTO manual for condition evaluation of bridge same as slab. Only one type of maintenance considers analyzed life-cycle cost because of characteristic maintenance pier.

Table 19 Expected cost of repair pier with 75 year service life

No. of Repair	**Corrosion rate 0.11mm/year**		**Corrosion rate 0.15mm/year**	
	Year	**Unit Cost (US\$/m^2)**	**Year**	**Unit Cost (US\$/m^2)**
1	16	39.365	12	49.697
2	30	17.411	23	26.180
3	44	7.701	33	14.619
4	58	3.406	43	8.163
5	72	1.507	53	4.588
6			63	2.545
7			73	1.421
Total		69.389		107.183



a.) Corrosion rate 0.11mm/year



b.) Corrosion rate 0.15mm/year

Figure 59 A time line for expected annual cost of repair concrete bridge deck

Figure 60 and Table 20 show the relationship between corrosion rates and expected total life-cycle costs of different maintenance strategies. It shows that the various maintenance concrete repair is the most effective maintenance action. Minor repair results in very significant improvement of total life-cycle cost when corrosion rate of the reinforced concrete is minor. Conversely, high corrosion rate of a major maintenance gives the better result.

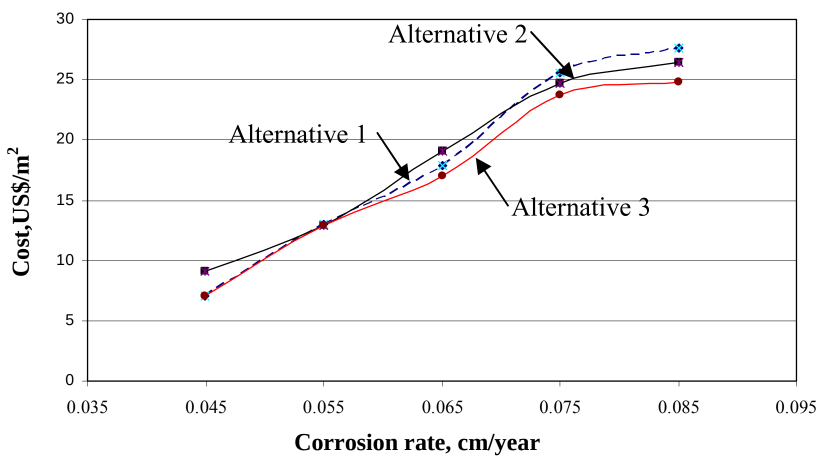


Figure 60 Relationship between corrosion rate and total costs for maintenance in slab

Table 20 Summary of expected total cost for maintenance slab with 75 year service life

Corrosion rate cm/year	Alternative 1: "Minor repair" Unit Cost (US$/m^2)	Alternative 2: "Major repair" Unit Cost (US$/m^2)	Alternative 3: "Various repair" Unit Cost (US$/m^2)
0.045	7.09	9.09	7.09
0.055	13.04	12.9	12.9
0.065	17.82	19.01	16.75
0.075	25.61	24.71	23.77
0.085	27.65	26.39	24.75

In Figure 61 and Table 21, the relationship between corrosion rate and expected total life-cycle costs for pier are compared. The result also shows repair strategy of reinforced concrete pier due to chloride induce corrosion.

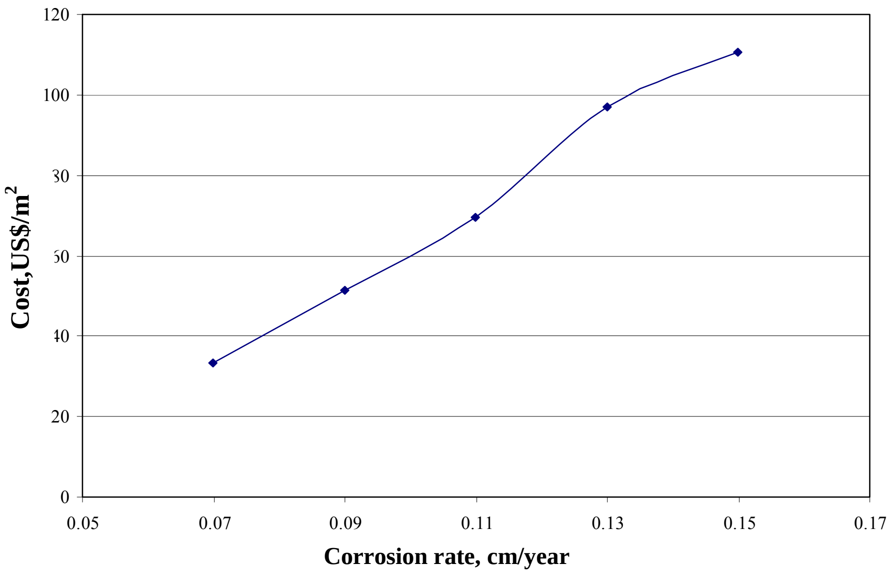


Figure 61 Relationship between corrosion rate and expect total life-cycle costs for pier

Table 21 Summary of expected total cost for maintenance pier with 75 year service life

Corrosion rate cm/year	**Alternative 1: "Major repair" Unit Cost (US$/m^2)**
0.07	33.454
0.09	51.576
0.11	69.389
0.13	96.959
0.15	107.183

CONCLUSION AND RECOMMENDATION

Conclusion

The results of the research reported in this paper are limited to a reinforced concrete bridge structure due to chloride-induced corrosion. Based on the structural reliability approach, the proposed deterioration model can incorporate uncertainties inherent in random variables in a reduction in load-carrying capacities due to a loss in cross-sectional area of the reinforcing steel bars. An application of the model is illustrated using the deteriorated reinforced concrete bridge in the southern part of Thailand.

The following conclusions are made based on the results of the study:

1. The proposed life cycle cost model that predicts the time to repair bridge decks subjected to chloride-induced reinforcement corrosion can be modified to incorporate the natural variability associated with concrete bridge, environmental exposure conditions, and reinforcement corrosion by using statistical computing techniques.

2. The effect of different maintenance was illustrated by comparing results of, minor repair, and major repair and various repairs for the concrete bridge deck. It was shown that various repair can provide an optimum option on the expected total cost of repair when corrosion rate is greater than 0.055cm/year.

3. The analysis procedure provide in this study can be used as a guideline for maintenance strategies. It should be emphasized that the procedure was intentionally developed for reinforced concrete bridges, subject to corrosion damage, in which moment is the dominant failure mode.

4. The MATLAB program developed in this study can be updated for site specific information to improve service life prediction and decision regarding repair and maintenance action of concrete bridge structure.

Recommendation

The following recommendations for future research are made based on the results of the study:

1. The value of the chloride initiation concentration can have a significant affect on the time to chloride initiation of concrete bridge structures. The accuracy of predictions of the time to chloride initiation will be limited until the value, or distribution, of the chloride initiation concentration is better defined. Research that investigates the chloride initiation concentration of field structures, would improve future predictions of the time to repair.

2. The optimization of bridge maintenance actions must combine both analytical models and the results obtained from non-destructive tests, visual inspections, and proactive health monitoring. In this study, a non-destructive tests and historical records from literatures on foreign countries are used. However, more accurate assessment and prediction of performance will be possible if the results provided by this model are updated using information on routine inspection.

3. There are some limitations in research study, indicating that further research is need. The analysis has been restricted to the flexural and compression capacity considerations. Functionality, considerations such as shear, cracking effect, bond and serviceability were not considered.

4. According to the calculation result, the chloride initiation of pier can have a significant affect on the deterioration model. The service life of concrete pier is short period due to the chloride diffused through the protective concrete cover. So, the type-5 portland cement should be strongly recommended for construction, of new bridge structure.

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APPENDICES

Appendix A

Data input program

```
%Script by Surattachon Chauytook
%Update June 2008
%To simulate the data input for analysis
%a vector of variable
% for concrete slab deck

function datainput_ti
        %Data input follow up:
        %Trevor J. Kirkpatricka, Richard E. Weyersa
        %Probabilistic model for the chloride-induced corrosion service life of
bridge decks

        %**** Time to initiation of concrete
                t_crack = 0;

                % ---Data for Bridge -------
                % Chloride Corrosion Initiation Concentration (kg/m^3)
                min_CC = 0.6;
                max_CC = 1.2;

                % Average and Std. Dev. of Surface Chloride Content (Co) (kg/m^3)
               mean_Co = 0.993;
                std_Co = 0.197*mean_Co;

                % Average and Std. Dev. of Diffustion Coefficient (Dc) (mm^2/year)
               mean_Dc = 18.164;
                std_Dc = 0.443*mean_Dc;

                % Average and Std. Dev. of Cover Depth (x) (mm)
                mean_x = 26;
                 std_x = 0.199*mean_x;

           %********* save data input
                     save data_chloride
```

```
%Script by Surattachon Chauytook
%Update June 2008
%To simulate the data input for analysis
%a vector of variable
% for concrete slab span 10m.

function datainput_bridge

        %Data input follow up:

        %Department of Rural Roads,Standard Drawing TC 4-101&2,Thailand
        %*** Materials Properties& Geometry Properties ****
        %*** Section Properties ***
                    s = 10;    %span lenght of bridge(m)
                   wi = 7;    %wide of bridge(m)
                   nl = 2;    %number of lane
                    b = 1 ;   %width of section(m)
                    h = 0.50; %thickness of section(m)
                cov_h = 0.019;%COV of thickness
                dis_h = 1;    %distribution(1:normal,2:longnor)

            %*** Concrete Properties ***
                   fc = 250;   %norminal of concrete strength (ksc)
              mean_fc = 250;   %mean of concrete strength (ksc)
               cov_fc = 0.166; %COV of concrete strength
               dis_fc = 1;     %distribution(1:normal,2:longnor)
                   ec = 240000;%moduls of elasticity of concrete(ksc)
                   wc = 0.4;   %water cement ratio
              mean_ec = 240000;%mean of ec
               cov_ec = 0.06;  %COV of ec
               dis_ec = 1;     %distribution(1:normal,2:longnor)

            %*** Steel Properties ***
                fy_sd = 4000;  %norminal of SD40(ksc)
           mean_fy_sd = 4614;  %mean of SD40(ksc)
            cov_fy_sd = 0.05; %cov of SD40
            dis_fy_sd = 2;     %distribution(1:normal,2:longnor)

                fy_sr = 2400;  %norminal of SR24(ksc)
           mean_fy_sr = 3467;  %mean of SD40(ksc)
            cov_fy_sr = 0.068; %cov of SD40
            dis_fy_sr = 2;     %distribution(1:normal,2:longnor)

                   es = 2100000;%Moduls of Elasticity of Steel(ksc)
              mean_es = 2110500;%mean of ec
               cov_es = 0.033;  %COV of ec
               dis_es = 1;      %distribution(1:normal,2:longnor)



            %*** compression steel ***
                  fy_c = fy_sr;     %norminal SD
             mean_fy_c = mean_fy_sr;%mean
              cov_fy_c = cov_fy_sr; %COV
              dis_fy_c = dis_fy_sr; %distribution(1:normal,2:longnor)

                dia_ci = 9;    %norminal RB
           mean_dia_ci = 9.09; %mean RB
             cov_diaci = 0.0099; %COV RB
             dis_diaci = 2;     %distribution(1:normal,2:longnor)
                spec_c = 0.25; %spacing of tension steel(m)

        %Data input follow up:
        %Anrzej S.Nowak 2007
        %Calibration of Bridge Code

            %******** Dead Load Model(Nowak 1993):D
               lamda_1 = 1.05; %bias factor of cast-in-place concrete:x1
               lamda_2 = 1.10; %bias factor of asphalt surface(75mm)
               lamda_3 = 0.00; %bias factor of miscellaneous
                cov_d1 = 0.10; %COV of cast-in-place concrete
```

```
%*** Section Properties ***
        dc = 0.035; %norminal concrete covering compression steel(m)
   mean_dc = 0.022; %mean of covering compression steel(m)
    cov_dc = 0.197;  %COV of dc
    dis_dc = 1;     %distribution(1:normal,2:longnor)

        dt = 0.025; %norminal concrete covering tension steel(m)
   mean_dt = 0.026; %mean of concrete covering tension steel(m)
    cov_dt = 0.199;  %COV of dt
    dis_dt = 1;     %distribution(1:normal,2:longnor)

%*** tension steel ***
      fy_t = fy_sd;     %norminal SD30
 mean_fy_t = mean_fy_sd;%mean
  cov_fy_t = cov_fy_sd; %COV
  dis_fy_t = dis_fy_sd; %distribution(1:normal,2:longnor)

    dia_ti = 25;    %norminal DB25(mm)
mean_dia_ti = 24.94; %mean DB25
 cov_diati = 0.006; %COV DB25
 dis_diati = 2;     %distribution(1:normal,2:longnor)
    spec_t = 0.09;  %spacing of tension steel(m)

    cov_d2 = 0.25; %COV of asphalt surface
    cov_d3 = 0.10; %COV of miscellaneous
    dis_d1 = 1;    %distribution(1:normal,2:longnor)
    dis_d2 = 1;    %distribution(1:normal,2:longnor)
    dis_d3 = 1;    %distribution(1:normal,2:longnor)

%******** Live load model(Nowak 1993):L
        mll = 1.3*22510; %moment of live load(kg-m)
   lamda_ll = 1.73;%bias factor of live load:x2
     cov_ll = 0.12; %COV of live load
     dis_ll = 1;    %distribution(1:normal,2:longnor)

         lf = 1.0;  %lanes factor
        imf = 0.18; %impact factor (dynamic fator)
    cov_imf = 0.8;  %COV of impact factor
    dis_imf = 1;    %distribution(1:normal,2:longnor)

%Data input follow up:
%Kim Anh T. Vu, Mark G.Stewart
%Structral reliability of concrete bridge including improved
%chloride-induced corrosion models
%Structural safety 22(2000)p.323-p.333

         Ri = 0.075; %norminal model error of icorr
    mean_Ri = 0.075; %mean odel error of icorr
     cov_Ri = 0.35; %COV of model error
     dis_Ri = 1;   %distribution(1:normal,2:longnor)
         gx = zeros(16,3);
```

```
 %--MEAN MOMENT--%       %--------STD---------%   %-----distribution----%

 %----------dead load
 gx(1,1) = lamda_1;      gx(1,2) = gx(1,1)*cov_d1;      gx(1,3) = dis_d1;
 gx(2,1) = lamda_2;      gx(2,2) = gx(2,1)*cov_d2;      gx(2,3) = dis_d2;
 gx(3,1) = lamda_3;      gx(3,2) = gx(3,1)*cov_d3;      gx(3,3) = dis_d3;
 gx(4,1) = h;            gx(4,2) = gx(4,1)*cov_h;       gx(4,3) = dis_h;
 %---------live load
 gx(5,1) = lamda_ll;     gx(5,2) = gx(5,1)*cov_ll;      gx(5,3) = dis_ll;
 gx(6,1) = imf;          gx(6,2) = gx(6,1)*cov_imf;     gx(6,3) = dis_imf;
 %---------Resistance
 gx(7,1) = mean_fc;      gx(7,2) = gx(7,1)*cov_fc;      gx(7,3) = dis_fc;
 gx(8,1) = mean_fy_t;    gx(8,2) = gx(8,1)*cov_fy_t;    gx(8,3) = dis_fy_t;
 gx(9,1) = mean_fy_c;    gx(9,2) = gx(9,1)*cov_fy_c;    gx(9,3) = dis_fy_c;
gx(10,1) = mean_dia_ci; gx(10,2) = gx(10,1)*cov_diaci; gx(10,3) = dis_diaci;
gx(11,1) = mean_dia_ti; gx(11,2) = gx(11,1)*cov_diati; gx(11,3) = dis_diati;
gx(12,1) = mean_dc;     gx(12,2) = gx(12,1)*cov_dc;    gx(12,3) = dis_dc;
gx(13,1) = mean_dt;     gx(13,2) = gx(13,1)*cov_dt;    gx(13,3) = dis_dt;
gx(14,1) = mean_ec;     gx(14,2) = gx(14,1)*cov_ec;    gx(14,3) = dis_ec;
gx(15,1) = mean_es;     gx(15,2) = gx(15,1)*cov_es;    gx(15,3) = dis_es;
%----------Corrosion
gx(16,1) = mean_Ri;     gx(16,2) = gx(16,1)*cov_Ri;    gx(16,3) = dis_Ri;

   %********* save data input
             save data_bridge
```

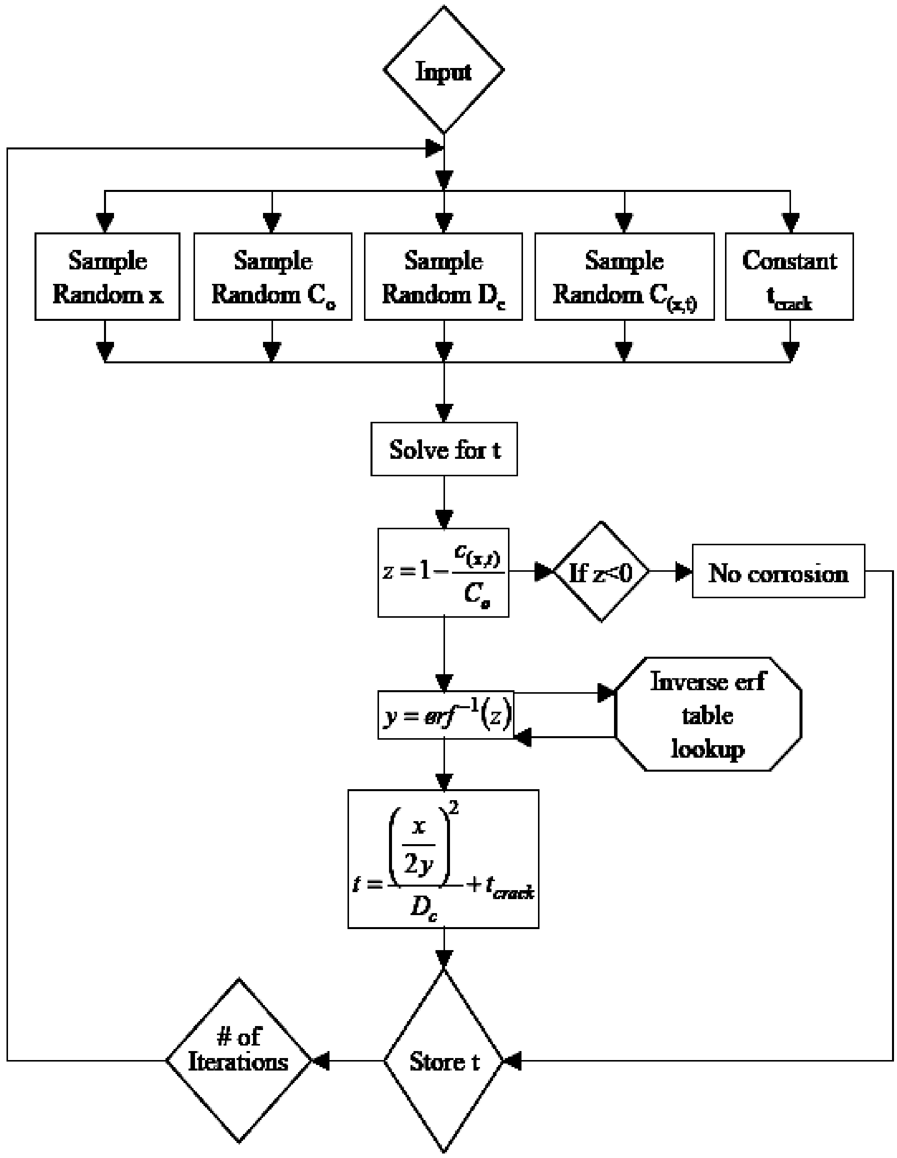
```
%Script by Surattachon Chauytook
%Update June 2008
%To simulate the data input for analysis
%a vector of variable
% for column

function datainput_ti
        %Data input follow up:
        %Trevor J. Kirkpatricka, Richard E. Weyersa
        %Probabilistic model for the chloride-induced corrosion servic life of bridge
column

        %**** Time to initiation of concrete
                t_crack = 0;

                % ---Data for Bridge -------
                % Chloride Corrosion Initiation Concentration (kg/m^3)
                min_CC = 0.6;
                max_CC = 1.2;

                % Average and Std. Dev. of Surface Chloride Content (Co) (kg/m^3)
               mean_Co = 23.31;
                std_Co = 0.197*mean_Co;

                % Average and Std. Dev. of Diffustion Coefficient (Dc) (mm^2/year)
               mean_Dc = 100.84;
                std_Dc = 0.443*mean_Dc;

                % Average and Std. Dev. of Cover Depth (x) (mm)
                mean_x = 70;
                 std_x = 0.182*mean_x;

                % chloride-induced corrosion models
                     Ri = 0.11; %norminal model error of icorr
                 std_Ri = 0.2*Ri; %COV of model error
                   wlim = 0.5;%crack width,(mm)


           %********* save data input
                     save data_chloride
```

```
%Script by Surattachon Chauytook
%Update June 2008
%To simulate the data input for analysis
%a vector of variable
% for column
function datainput_column

        %Data input follow up:
        %Department of Rural Roads,Standard Drawing TC 2-203,Thailand
        %*** Materials Properties& Geometry Properties ****
        %*** Section Properties ***
                    s = 10;   %span lenght of bridge(m)
                   sw = 2.00;  %span column to column(bay),m
                   wi = 7;    %wide of bridge(m)
                   nl = 2;    %number of lane
                    b = 1 ;   %width of section(m)
                   hh = 5.00;
                   bb = 0.80; %width of section beam(m)
                   hb = 0.80; %width of section depth(m)
                   bc = 0.40; %width of column(m)
                   wc = 0.40; %width of column(m)
                    h = 0.50; %thickness of section(m)
                cov_h = 0.016;%COV of thickness
                dis_h = 1;    %distribution(1:normal,2:longnor)

            %*** Concrete Properties ***
                   fc = 250;  %norminal of concrete strength (ksc)
              mean_fc = 250;  %mean of concrete strength (ksc)
               cov_fc = 0.166;%COV of concrete strength
               dis_fc = 1;    %distribution(1:normal,2:longnor)

            %*** Steel Properties ***
                fy_sd = 4000; %norminal of SD40(ksc)
           mean_fy_sd = 4614; %mean of SD40(ksc)
            cov_fy_sd = 0.042;%cov of SD40
            dis_fy_sd = 2;    %distribution(1:normal,2:longnor)

             %*** compression steel ***
                  fy_t = fy_sd;     %norminal SD40
             mean_fy_t = mean_fy_sd;%mean
              cov_fy_t = cov_fy_sd; %COV
              dis_fy_t = dis_fy_sd; %distribution(1:normal,2:longnor)

                dia_ci = 20;    %norminal DB20(mm)
           mean_dia_ci = 19.98; %mean DB25
             cov_diaci = 0.006; %COV DB25
             dis_diaci = 2;     %distribution(1:normal,2:longnor)
               spec_ci = 0.09;  %spacing of steel(m)

            %*** Section Properties ***
                    dc = 0.05;%norminal concrete covering compression steel(m)
               mean_dc = 0.07;%mean of covering compression steel(m)
                cov_dc = 0.0182; %COV of dc
                dis_dc = 1;    %distribution(1:normal,2:longnor)
                    ns = 4;    %amount of steel reinforce concrete

        %Data input follow up:
        %Anrzej S.Nowak 2007
        %Calibration of Bridge Code

            %******** Dead Load Model(Nowak 1993):D
               lamda_1 = 1.05; %bias factor of cast-in-place concrete:x1
               lamda_2 = 1.10; %bias factor of asphalt surface(75mm)
               lamda_3 = 0.00; %bias factor of miscellaneous
                cov_d1 = 0.10; %COV of cast-in-place concrete
                cov_d2 = 0.25; %COV of asphalt surface
                cov_d3 = 0.10; %COV of miscellaneous
                dis_d1 = 1;    %distribution(1:normal,2:longnor)
                dis_d2 = 1;    %distribution(1:normal,2:longnor)
                dis_d3 = 1;    %distribution(1:normal,2:longnor)
```

```
    %******** Live load model(Nowak 1993):L
            Pll = 1.3*4000; %moment of live load(kg-m)
      lamda_Pll = 1.50; %bias factor of live load:x2
        cov_Pll = 0.12; %COV of live load
        dis_Pll = 1;    %distribution(1:normal,2:longnor)

            imf = 0.15; %impact factor (dynamic fator)
        cov_imf = 0.8;  %COV of impact factor
        dis_imf = 1;    %distribution(1:normal,2:longnor)

%Data input follow up:
%Kim Anh T. Vu, Mark G.Stewart
%Structral reliability of concrete bridge including improved
%chloride-induced corrosion models
%Structural safety 22(2000)p.323-p.333

             Ri = 0.11; %norminal model error of icorr
        mean_Ri = 0.11; %mean odel error of icorr
         cov_Ri = 0.2; %COV of model error
         dis_Ri = 1;   %distribution(1:normal,2:longnor)

             gx = zeros(16,3);

 %--MEAN MOMENT--%       %--------STD----------%   %-----distribution----%
 %----------dead load
 gx(1,1) = lamda_1;      gx(1,2) = gx(1,1)*cov_d1;      gx(1,3) = dis_d1;
 gx(2,1) = lamda_2;      gx(2,2) = gx(2,1)*cov_d2;      gx(2,3) = dis_d2;
 gx(3,1) = lamda_3;      gx(3,2) = gx(3,1)*cov_d3;      gx(3,3) = dis_d3;
 gx(4,1) = h;            gx(4,2) = gx(4,1)*cov_h;       gx(4,3) = dis_h;
 gx(5,1) = sw;           gx(5,2) = gx(5,1)*cov_h;       gx(5,3) = dis_h;
 gx(6,1) = bb;           gx(6,2) = gx(6,1)*cov_h;       gx(6,3) = dis_h;
 gx(7,1) = hb;           gx(7,2) = gx(7,1)*cov_h;       gx(7,3) = dis_h;
 gx(8,1) = bc;           gx(8,2) = gx(8,1)*cov_h;       gx(8,3) = dis_h;
 gx(9,1) = wc;           gx(9,2) = gx(9,1)*cov_h;       gx(9,3) = dis_h;
 %---------live load
gx(10,1) = lamda_Pll;   gx(10,2) = gx(10,1)*cov_Pll;    gx(10,3) = dis_Pll;
gx(11,1) = imf;         gx(11,2) = gx(11,1)*cov_imf;    gx(11,3) = dis_imf;
%---------Resistance
gx(12,1) = mean_fc;     gx(12,2) = gx(12,1)*cov_fc;     gx(12,3) = dis_fc;
gx(13,1) = mean_fy_sd;  gx(13,2) = gx(13,1)*cov_fy_sd;  gx(13,3) = dis_fy_sd;
gx(14,1) = mean_dia_ci; gx(14,2) = gx(14,1)*cov_diaci;  gx(14,3) = dis_diaci;
%----------Corrosion
gx(15,1) = mean_Ri;     gx(15,2) = gx(15,1)*cov_Ri;    gx(15,3) = dis_Ri;
gx(16,1) = mean_dc;     gx(16,2) = gx(16,1)*cov_dc;    gx(16,3) = dis_dc;

   %********* save data input
             save data_column
```

Appendix B

Instruction program set for chloride initiation



Appendix Figure B1 Flow chart to determine time for corrosion

```
%Script by Surattachon Chauytook
%Update November 2008
%To simulate the time to initiation of concrete
%Return a Time to initiation of concrete
%Probabilistic model for chloride-induced corrosion of bridge
%simulate based on Fick's second law
%Where: C(x,t) = chloride concentration at depth and time
%       Co = surface chloride concentration
%       Dc = apparent diffusion coefficient
%        t = time for diffusion
%        x = concrete cover depth
%      erf = statistical error function
%Calculation follow up:
%Trevor J. Kirkpatricka, Richard E. Weyersa
%Probabilistic model for the chloride-induced corrosion service life of bridge decks
%Received 15 August 2001; accepted 12 June 2002
%Output tini(time to initiation)

function[tini] = initiation

         %**** Time to initiation of concrete
%% STEP1 Load Data input file
clc
clear
                load data_chloride

%% STEP2 Amount number for random Vriables
                     n = 2^18;
                 nbins = 2^9;

%% STEP3 Random Vriables
                % ---Data for Bridge -------
          % Chloride Corrosion Initiation Concentration, (kg/m^3)
                  CC = unifrnd(min_CC,max_CC,n,1);

          % Average and Std. Dev. of Surface Chloride Content, (kg/m^3)
                  Co = gamrnd((mean_Co/std_Co)^2,std_Co^2/mean_Co,[n 1]);

          % Average and Std. Dev. of Diffustion Coefficient, (mm^2/year)
                  Dc = gamrnd((mean_Dc/std_Dc)^2,std_Dc^2/mean_Dc,[n 1]);

          % Average and Std. Dev. of Cover Depth, (mm)
                  x = normrnd(mean_x,std_x,[n 1]);

%% STEP4 Evaluate time to initiation of concrete random variable
                     z = ones(size(x)) - (CC./Co);
                     y = erfinv(z(find(CC <= Co)));
                    xx = x(find(CC <= Co));
                    DD = Dc(find(CC <= Co));
                     t = (xx./(2*y)).^2./DD + t_crack;

%% STEP5 Find time to initiation of concrete at Probability 50%
           data_prctile = round(prctile(t,2.50));

%% STEP6 Results
          %**** Time to initiation of concrete
                 tini = data_prctile;

%% STEP7 Save Resluts
            save ('op_initiation')
```

Appendix C

Instruction program set for deterioration

```
%Script by Surattachon Chauytook
%Update November 2008
%To simulate the bridge managmant system
%Probabilistic model for chloride-induced corrosion of bridge
%a vector of random variable

function[index]= run

index = zeros(75,1);

%% STEP1 Load data input
    datainput_ti;     %Data input file for chloride data
    datainput_bridge; %Data input file for bridge data

%% STEP2 Calculate according AASHTO code
    aashto; %Calculate according AASHTO

%% STEP3 Find time to initiation
    ti = initiation; %Find time to initiation

%% STEP4 Find relability index no corrosion
    deadload; %Find dead load function
    liveload; %Find live load function
    resist0;  %Find resistance no corrosion induced
    limitf0;  %Defind limit state function
    index(1:ti,1) = afosm0;%Find relability index used AFOSM Method

%% STEP5 Find relability index due to corrosion

    for t = ti+1:65
        resist1(t,ti);
        limitf1;
        index(t,1) = afosm1  ;
        if index < 1
        continue
        end
    end

%% STEP10 SAVE Results
     save ('output_slab')
```

```
%Script by Surattachon Chauytook
%Update November 2008
%To simulate the limit state function
%output limit state function and data of X
%input time to analysis
%following /Nowak /AASHTO 1996 /ACI318
%Probabilistic model for chloride-induced corrosion of bridge
%a vector of random variable

function deadload
load data_bridge
syms lamda_1 lamda_2 lamda_3 h x1 x2 x3 x4

%******** Dead Load Model(Nowak 1993):D
          mdl_1 = 2400*b*h*s^2/8*lamda_1;     %moment of cast-in-place concrete(kg-m)
          mdl_2 = 2300*0.075*b*s^2/8*lamda_2 ;%moment of asphalt surface(75mm)(kg-m)
          mdl_3 = 0*lamda_3;                  %moment of miscellaneous(kg-m)

        %------ Dead Load Model-------
              d = mdl_1 + mdl_2 + mdl_3;
              D = subs(d,{'lamda_1','lamda_2','lamda_3','h'},{x1,x2,x3,x4});

%******* Save Data ********************
            save ('op_dead','D')



%Script by Surattachon Chauytook
%Update November 2008
%To simulate the limit state function
%output limit state function and data of X
%input time to analysis
%following Nowak AASHTO 1996 ACI318
%Probabilistic model for chloride-induced corrosion of bridge
%a vector of random variable

function liveload

%% STEP1 Load Data Input File
load data_bridge

%% STEP2 Evaluate Live Load According AASHTO 1996
syms imf lamda_ll x5 x6
%******** Live load model(AASTHO-1996 HS-20,Nowak 1993)
              cc = 4+0.06*(s/0.3048);
             gdf = cc*0.3048;%distribution width factor

          %-------Live load model-----%
               ll = mll*(1+imf)*lamda_ll*lf/gdf;

%% STEP3 Substitute Vriable in function xi
               L = subs(ll,{'lamda_ll','imf'},{x5,x6});

%% STEP4 Save Results
        %******* Save Data
             save ('op_live','L')
```

```
%Script by Surattachon Chauytook
%Update June 2008
%To simulate the limit state function
%output limit state function and data of X
%input time to analysis
%following Nowak AASHTO 1996 ACI318
%Probabilistic model for chloride-induced corrosion of bridge
%a vector of random variable

function resist1(t,ti)

%% STEP1 Load Data input file
load data_bridge

%% STEP2 Define Resistance Model
%*** Norminal Resistance Model(Stength Method Design)******
        %*** corrosion rate  ***
           % icorr = (37.8*(1-wc)^(-1.64))/(dc*100)*0.85*(t-ti)^(-0.29);%current
density
            lamda = Ri;%0.0116*icorr*Ri;

        %*** tension steel ***
             dia_t = dia_ti-2*lamda*(t-ti);%diameter of steel bar at time
              as_t = pi*(dia_t/10)^2/4*b/spec_t;%area of steel(cm2)

        %*** compression steel ***
             dia_c = dia_ci-2*lamda*(t-ti);%diameter of steel bar at time
              as_c = pi*(dia_c/10)^2/4*b/spec_c;%area of steel(cm2)

       %Calculation Moment Norminal Resistance Capacity(kg-m)
                    if fc < 300
                        beta1 = 0.85;
                    else
                        beta1 = 0.85-0.0008*(fc-300);
                    end
                   a = ((as_t-as_c)*fy_t)/(0.85*fc*b*100);
                es_c = 0.003*(a-beta1*dc*100)/a;
                es_t = 0.003*(beta1*(h-dt)*100-a)/a;

           %check condition stress in steel bar
                es_cr = fy_c/es;
                es_tr = fy_t/es;


               if es_c > es_cr && es_t > es_tr
                    r = mn1(t,ti);
               elseif es_c < es_cr && es_t > es_tr
                    r = mn1(t,ti);
                    %r = mn2(beta1,a,t,ti);
               else
                    r = mn1(t,ti);
                    %r = mn3(beta1,a,t,ti);
               end

%% STEP3 Substitute Variable Resistance function in xi* function
            syms x7 x8 x9 x10 x11 x12 x13 x14 x15 x16
                    R =
subs(r,{'fc','fy_t','fy_c','dia_ci','dia_ti','dc','dt','ec','es','Ri'},
{x7,x8,x9,x10,x11,x12,x13,x14,x15,x16});

%% STEP4 Save Result
            save ('op_resis1','R')
```

```
%Script by Surattachon Chauytook
%Update June 2008
%To simulate the limit state function
%output limit state function and data of X
%input time to analysis
%following Nowak AASHTO 1996 ACI318
%Probabilistic model for chloride-induced corrosion of bridge
%Output limit state function in variable

function limitf1

%% STEP1 Load Data input file
load op_dead
load op_live
load op_resis1
load data_bridge

%% STEP2 Define Limit State Function
%****** Define limit state function ******
            g = R-D-L;

%% STEP3 Save Result
%******* Save Data
            save ('limitf1','g','gx')

%Script by Surattachon Chauytook
%Update June 2008
%To simulate the structural reliability analysis
%based on advance first order second moment method(AFOSM)
%Probabilistic model for chloride-induced corrosion of bridge

%***********************************************************
%*** Alogorithm AFOSM Method for Lognormal Random Variable
%***********************************************************

function[b_index] = afosm1

%% STEP1 Load Limmit State Function
    %****** Define limit state function ******
    load limitf1

%% STEP2 Evaluate d(gx)/dxi
            gxf(1,1) = diff(g,'x1');
            gxf(2,1) = diff(g,'x2');
            gxf(3,1) = diff(g,'x3');
            gxf(4,1) = diff(g,'x4');
            gxf(5,1) = diff(g,'x5');
            gxf(6,1) = diff(g,'x6');
            gxf(7,1) = diff(g,'x7');
            gxf(8,1) = diff(g,'x8');
            gxf(9,1) = diff(g,'x9');
           gxf(10,1) = diff(g,'x10');
           gxf(11,1) = diff(g,'x11');
           gxf(12,1) = diff(g,'x12');
           gxf(13,1) = diff(g,'x13');
           gxf(14,1) = diff(g,'x14');
           gxf(15,1) = diff(g,'x15');
           gxf(16,1) = diff(g,'x16');
```

```
%% STEP3 Evaluate Mean and STD Parameter
          %Mean and Std of Resistance Capacity
              for i = 1:16
                  if gx(i,3) == 2%%A lognormal distribution parameters
                       gxx(i,1) = log((gx(i,1)^2)/(sqrt(gx(i,2)^2+gx(i,1)^2)));
 %mean of lognormal distribution
                       gxx(i,2) = sqrt(log(gx(i,2)^2/(gx(i,1)^2)+1));
%Standard Deviation of lognormal distribution


                  elseif gx(i,3) == 1      %A lognormal distribution parameters
                       gxx(i,1) = gx(i,1); %mean of lognormal distribution defined by
matlab
                       gxx(i,2) = gx(i,2);
%Standard Deviation of lognormal distribution defined by matlab

                  else
                      fprintf
(1,'*** distribution at gx %2.0f error assume normal distribution***\n',i);
                       gxx(i,1) = gx(i,1); %mean of normal distribution
                       gxx(i,2) = gx(i,2); %Standard Deviation of normal distribution
                  end
              end

%% STEP4 Assume Parameter
        %assume condition
            b_index = 3.0;
          diff_beta = 1;
                 loop = 0;

        %******* Start Loop to find Reliability Index(beta)********
            while diff_beta > 0.01
                loop = loop+1;
                sum_grad2 = 0;

%% STEP5 Evaluate Normal Distribution Parameter
                for i = 1:16
                      if gx(i,3) == 2
                        %Find Equivalent Normal Distribution Parameter for R
                        xr(i,2) = (normpdf(norminv(logncdf(gx(i,1),gxx(i,1)
,gxx(i,2)),0,1),0,1))/lognpdf(gx(i,1),gxx(i,1),gxx(i,2));%Standard Deviation
                        xr(i,1) = gx(i,1)- xr(i,2)*(norminv(logncdf(gx(i,1),
gxx(i,1),gxx(i,2)),0,1));%mean
                      else
                        %For normal distribution
                        xr(i,2) = gxx(i,2); %Standard Deviation normal
                        xr(i,1) = gxx(i,1); %mean normal
                      end

%% STEP6 Evaluate Substitute Design Point in dg(xi*)/dxi
                        fx(i,1) =
subs(gxf(i,1),{'x1','x2','x3','x4','x5','x6','x7','x8','x9','x10','x11','x12','x13',
'x14','x15','x16'},{gx(1,1),gx(2,1),gx(3,1),gx(4,1),gx(5,1),gx(6,1),gx(7,1),
gx(8,1),gx(9,1),gx(10,1),gx(11,1),gx(12,1),gx(13,1),gx(14,1),gx(15,1),gx(16,1)});
                      grad(i,1) = fx(i,1)*xr(i,2);%dg(xi*)/dxi*std
                      sum_grad2 = sum_grad2+grad(i,1)^2;
                end

%% STEP7 Obtain New Design Point ui*=-aplhai.Beta in Terms of Unknow Beta
                %Find Reliability Index
                    syms betaa
                        for i = 1:16
                            alpha(i,1) = grad(i,1)/(sum_grad2)^0.5;
                                u(i,1) = -1*alpha(i,1)*betaa;
                               gu(i,1) = xr(i,1)+u(i,1)*xr(i,2);
                        end
```

```
%% STEP8 Substitute g(xi*)->g(ui*) in term of Unknow Beta
                 %limit state function of u
                      fu = subs(g,{'x1','x2','x3','x4','x5','x6','x7',
'x8','x9','x10','x11','x12','x13','x14','x15','x16'},
{gu(1,1),gu(2,1),gu(3,1),gu(4,1),gu(5,1),gu(6,1),gu(7,1),gu(8,1),gu(9,1)
,gu(10,1),gu(11,1),gu(12,1),gu(13,1),gu(14,1),gu(15,1),gu(16,1)});

%% STEP9 Solve for Beta given that g(ui*)=0
                %Reliability Index
                       b_index_data = solve(fu);
                        b_index_new = double(b_index_data(1,1));


%% STEP10 Evaluate Diff Beta
                 %Check differance beta index old & new
                          diff_beta = abs(b_index_new-b_index);
                            b_index = b_index_new;

%% STEP11 Use New Beta from Step9 to get Update Value of ui* and xi* (New Design
Point)
                      for i = 1:16
                           ur(i,1) = -1*alpha(i,1)*b_index;
                           gx(i,1) = double(xr(i,1)+ur(i,1)*xr(i,2)); % New Design
Point
                      end

%% STEP12 Repeat Step5 to Step11 Until Beta Converges or ui* Changes are
Insignificant
            end

%% STEP13 Save Resluts
            save ('op_afosm')
```

Appendix D

Instruction program set for optimization

```
%Script by Surattachon Chauytook
%Update August 2008
%input time to analysis
%following EVENT TREE
%Probabilistic model for chloride-induced corrosion of bridge
%a vector of random variable

function loop
tic %time clock
stp = 2;%step 2 year
slope = 0.077;%slope for deterioration
st = 76/stp ;
bi = zeros(st,16);
  maint = [0;0.7;2;1];%maintenance update
  costt = [0;0.25;0.5;1];%cost for maintenance function
costmin = 100;
  bimin = bi;
%column 1:year
%column 2:type of maintenance
%column 3:maintenance yes/no(1/0)
%column 4:sum maintenance
%column 5:reliability index
%column 6:reliability index update
%column 7:cost of maintenance
%column 8:sum cost of maintenance
%column 9:maintenance type2 yes/no(1/0)
%column 10:maintenance type3 yes/no(1/0)
%column 11:maintenance type4 yes/no(1/0)
%column 12:sum maintenance type2 yes/no(1/0)
%column 13:sum maintenance type3 yes/no(1/0)
%column 14:sum maintenance type4 yes/no(1/0)
%column 15:code for break
%column 16:slope of deterioration

rindex = 4.573;%reliability index from reliability analysis
tini = 28;%time to initiation
tn = round(tini/stp)+1;
j = 0;
bi(tn+1:st+1,16) = slope*stp;
bi(st+1,16) = slope;
%bi(st+1,16) = slope;


%%
        for i = 1:st
          bi(i+1,1) = bi(i,1)+stp;
         bi(st+1,1) = 75;
          if bi(i,1) <= tini
              bi(i,2) = 1;
              bi(i,3) = 0;
              bi(i,4) = 0;
              bi(i,5) = rindex;
              bi(i,6) = rindex;
              bi(i,7) = 0;
              bi(i,8) = 0;
              bi(i,9) = 0;
             bi(i,10) = 0;
             bi(i,11) = 0;
             bi(i,12) = 0;
             bi(i,13) = 0;
             bi(i,14) = 0;
             bi(i,15) = 1;
             %bi(i,16) = 0;
                    j = j+1;
          end
        end
```

```
%% Optimization 5
for x1 = 1:4 % at year 30
    i = j+1;
    bi(i,2) = x1; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x2 = 1:4 % at year 32
    i = j+2;
    bi(i,2) = x2; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x3 = 1:4 % at year 34
    i = j+3;
    bi(i,2) = x3; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x4 = 1:4 % at year 36
    i = j+4;
    bi(i,2) = x4; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x5 = 1:4 % at year 38
    i = j+5;
    bi(i,2) = x5; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x6 = 1:4 % at year 40
     i = j+6;
    bi(i,2) = x6; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x7 = 1:4 % at year 42
    i = j+7;
    bi(i,2) = x7; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x8 = 1:4 % at year 44
    i = j+8;
    bi(i,2) = x8; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x9 = 1:4 % at year 46
    i = j+9;
    bi(i,2) = x9; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
```

```
for x10 = 1:4 % at year 48
    i = j+10;
    bi(i,2) = x10; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x11 = 1:4 % at year 50
    i = j+11;
    bi(i,2) = x11; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x12 = 1:4 % at year 52
    i = j+12;
    bi(i,2) = x12; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x13 = 1:4 % at year 54
     i = j+13;
    bi(i,2) = x13; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x14 = 1:4 % at year 56
     i = j+14;
    bi(i,2) = x14; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x15 = 1:4 % at year 58
     i = j+15;
    bi(i,2) = x15; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x16 = 1:4 % at year 60
    i = j+16;
    bi(i,2) = x16; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x17 = 1:4 % at year 62
    i = j+17;
    bi(i,2) = x17; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x18 = 1:4 % at year 64
    i = j+18;
    bi(i,2) = x18; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
```

```
for x19 = 1:4 % at year 66
    i = j+19;
    bi(i,2) = x19; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x20 = 1:4 % at year 68
    i = j+20;
    bi(i,2) = x20; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x21 = 1:4 % at year 70
     i = j+21;
    bi(i,2) = x21; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x22 = 1:4 % at year 72
    i = j+22;
    bi(i,2) = x22; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x23 = 1:4 % at year 74
    i = j+23;
    bi(i,2) = x23; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end
for x24 = 1:4 % at year 76
    i = j+24;
    bi(i,2) = x24; %type of maintenance
         bi = indextable(bi,i,maint,rindex,costt,slope,stp,costmin);
    if bi(i,15) == 0
        continue
    end

%******** for minimum cost
        if costmin > bi(st,8)
           costmin = bi(st,8);
             bimin = bi;
        else
           costmin = costmin;
             bimin = bi;
        end
```

```
end
end
end
end
end
end
end
end
end
end
end
end
end
end
end
end
end
end
end
end
end
end
end
end
%******* Save Data
            save ('op_index')
            toc;
            t= toc;
            fprintf (1,'Time to Analysis %5.2f sec.\n ',t);
```

```
%Script by Surattachon Chauytook
%Update August 2008
%input time to analysis
%following EVENT TREE
%Probabilistic model for chloride-induced corrosion of bridge
%a vector of random variable
%///////////////////////////////////////////////////////
%column 1:year
%column 2:type of maintenance
%column 3:maintenance yes/no(1/0)
%column 4:sum maintenance
%column 5:reliability index
%column 6:reliability index update
%column 7:cost of maintenance
%column 8:sum cost of maintenance
%column 9:maintenance type2 yes/no(1/0)
%column 10:maintenance type3 yes/no(1/0)
%column 11:maintenance type4 yes/no(1/0)
%column 12:sum maintenance type2 yes/no(1/0)
%column 13:sum maintenance type3 yes/no(1/0)
%column 14:sum maintenance type4 yes/no(1/0)
%column 15:code for break
%column 16:slop of deterioration
%///////////////////////////////////////////////////////

function[bi] = indextable(bi,i,maint,rindex,costt,slope,stp,costmin)
%bi = indextable(bi,i,maint,rindex,costt,slope,stp);

     bi(i,3) = main(bi(i,2));%maintenance yes/no
    bi(:,16) = slop(bi(i,3),bi(:,16),i);%deterioration
     bi(i,4) = smai(bi(i-1,4),bi(i,3));%sum maintenance
     bi(i,5) = inde(bi(i-1,6),bi(i,16));%relability index
     bi(i,6) = uind(bi(i,5),bi(i,2),maint,rindex);%relability index update
     bi(i,7) = cost(bi(i,2),costt,bi(i,1));%cost of maintenance
     bi(i,8) = scos(bi(i-1,8),bi(i,7));%sum cost of maintenance
     bi(i,9) = main2(bi(i,2));%maintenance type2 yes/no
    bi(i,10) = main3(bi(i,2));%maintenance type3 yes/no
    bi(i,11) = main4(bi(i,2));%maintenance type4 yes/no
    bi(i,12) = smai(bi(i-1,9),bi(i-1,12));%sum maintenance type2
    bi(i,13) = smai(bi(i-1,10),bi(i-1,13));%sum maintenance type3
    bi(i,14) = smai(bi(i-1,11),bi(i-1,14));%sum maintenance type4
    bi(i,15) = code(bi(i-
1,3),bi(i,3),bi(i,4),bi(i,5),bi(i,14),bi(:,3),bi(i,8),costmin,i);%code for break
```

```
%Script by Surattachon Chauytook
%Update August 2008
%input time to analysis
%following EVENT TREE
%Probabilistic model for chloride-induced corrosion of bridge
%a vector of random variable

function[op] = main(type)

 if type > 1
     op = 1;
 else
     op = 0;
 end

%Script by Surattachon Chauytook
%Update August 2008
%sum cost for maintenance

function[op] = slop(typ,mnew,i)
%bi(:,16) = slop(bi(i,3),bi(:,16),i);%deterioration
            op = mnew;
        if typ == 4
            if i+7 < 39
               op(i:i+7,:) = 0;
           % else
           %     op(i:38,:) = 0;
            end
        else
                op = mnew;
        end

%Script by Surattachon Chauytook
%Update August 2008
%sum of number maintenance

function[op] = smai(mold,mnew)
% bi(i,4) = smai(bi(i-1,4),bi(i-1,3))
     op = mold+mnew;
```



```
%Script by Surattachon Chauytook
%Update August 2008
%sum of number maintenance

function[op] = smai(mold,mnew)
% bi(i,4) = smai(bi(i-1,4),bi(i-1,3))
     op = mold+mnew;
```



```
%Script by Surattachon Chauytook
%Update August 2008
%calculation of reliability index

function[op] = inde(mold,deter)
%i(i,5) = inde(bi(i-1,6),bi(i,16));%relability index

     op = mold-deter;
```

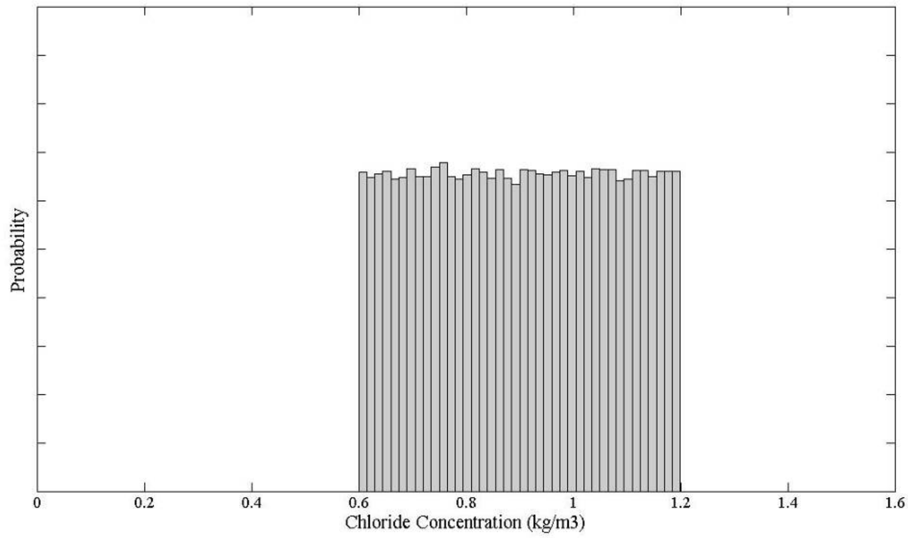
```
%Script by Surattachon Chauytook
%Update August 2008
%This script for update reliability index

function[op] = uind(mold,typ,maint,rindex)
    %bi(i,6) = uind(bi(i,5),bi(i,2),maint,rindex);%relability index update

         if typ == 1 %Do noting
         op = mold;
     elseif typ == 2 %minor maintenance
         op = mold + maint(2,1);
     elseif typ == 3 %major maintenance
         op = mold + maint(3,1);
     elseif typ == 4 %Replacement
         op = mold + maint(4,1);
         end

         if op > rindex %reliability index maximum allowable
         op = rindex;
         end
%Script by Surattachon Chauytook
%Update August 2008
%cost for type maintenance

function[co] = cost(typ,costt,t)
%bi(i,7) = cost(bi(i,2),costt)

         if typ == 1 %cost for do noting
         co = costt(1,1)/(1+0.02)^t;
     elseif typ == 2 %cost for minor maintenance
         co = costt(2,1)/(1+0.02)^t;
     elseif typ == 3 %cost for major maintenance
         co = costt(3,1)/(1+0.02)^t;
     elseif typ == 4 %cost for replacement
         co = costt(4,1)/(1+0.02)^t;
         end

%Script by Surattachon Chauytook
%Update August 2008
%sum cost for maintenance

function[op] = scos(mold,mnew)
    %bi(i,8) = scos(bi(i-1,8),bi(i,7))

        op = mold + mnew;

%Script by Surattachon Chauytook
%Update August 2008
%input time to analysis
%following EVENT TREE
%Probabilistic model for chloride-induced corrosion of bridge
%a vector of random variable

function[op] = main2(type)

 if type == 2
     op = 1;
 else
     op = 0;
 end
```

```
%Script by Surattachon Chauytook
%Update August 2008
%input time to analysis
%following EVENT TREE
%Probabilistic model for chloride-induced corrosion of bridge
%a vector of random variable

function[op] = main3(type)

 if type == 3
     op = 1;
 else
     op = 0;
 end

%Script by Surattachon Chauytook
%Update August 2008
%input time to analysis
%following EVENT TREE
%Probabilistic model for chloride-induced corrosion of bridge
%a vector of random variable

function[op] = main4(type)

 if type == 4
     op = 1;
 else
     op = 0;
 end

%Script by Surattachon Chauytook
%Update August 2008
%sum of number maintenance

function[op] = smai(mold,mnew)
% bi(i,4) = smai(bi(i-1,4),bi(i-1,3))
     op = mold+mnew;
```

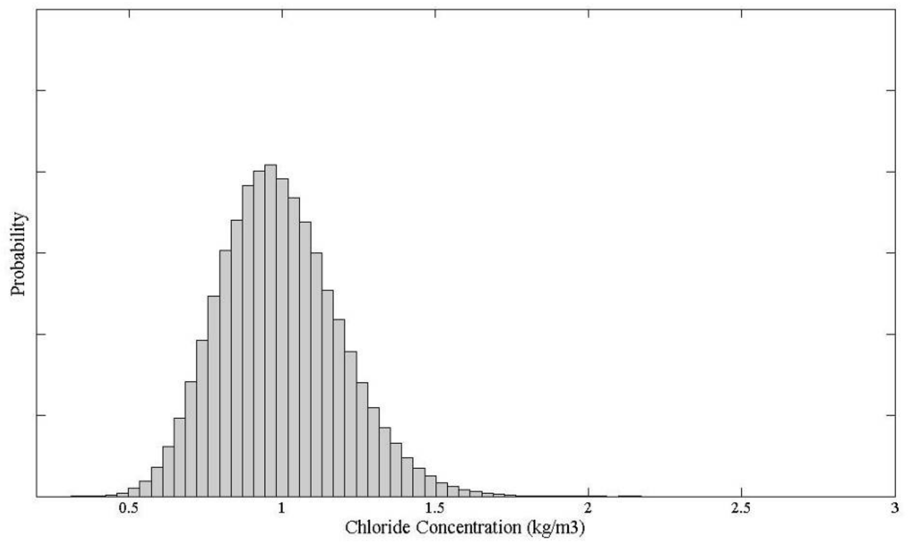
```
%Script by Surattachon Chauytook
%Update August 2008
%code for break
%column 1:year
%column 2:type of maintenance
%column 3:maintenance yes/no(1/0)
%column 4:sum maintenance
%column 5:reliability index
%column 6:reliability index update
%column 7:cost of maintenance
%column 8:sum cost of maintenance
%column 9:maintenance type2 yes/no(1/0)
%column 10:maintenance type3 yes/no(1/0)
%column 11:maintenance type4 yes/no(1/0)
%column 12:sum maintenance type2 yes/no(1/0)
%column 13:sum maintenance type3 yes/no(1/0)
%column 14:sum maintenance type4 yes/no(1/0)
%column 15:code for break

function[op] = code(cp1,cp1a,cp2,cp3,cp4,cp5,cp6,cp7,i)
  % bi(i,15) = code(bi(i-
1,3),bi(i,3),bi(i,4),bi(i,5),bi(i,14),bi(:,3),bi(i,8),costmin,i);%code for break
       %cp8 = sum(cp5(i-8:i,1));
         if cp2 > 8 %no maintenance
         op = 0;
     elseif cp1 == 1 & cp1a == 1
         op = 0;
     elseif cp3 < 2
         op = 0;
     elseif cp4 > 2
         op = 0;
     elseif cp5 == 1 %& cp8 > 1
         op =0;
     elseif cp6 > cp7
         op =0;
     else
         op = 1;
        end
```

Appendix E

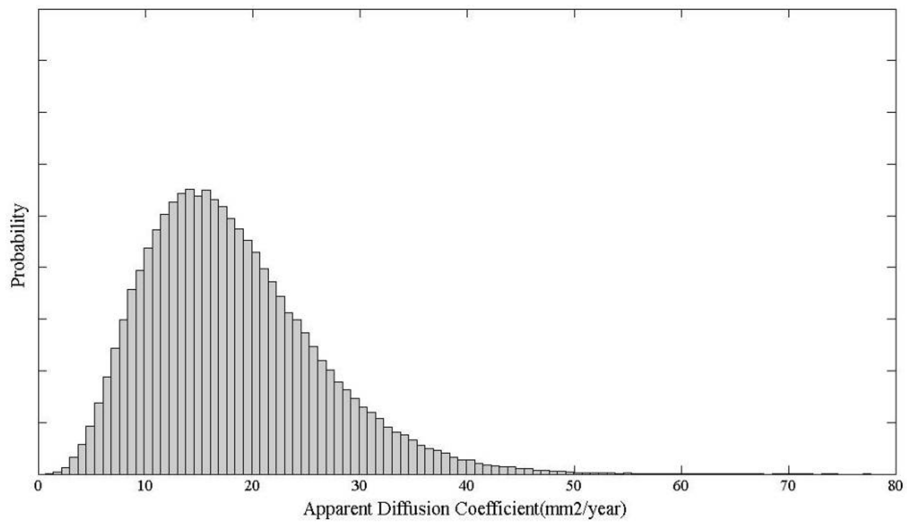
Probability distribution of parameter for time to corrosion initiation



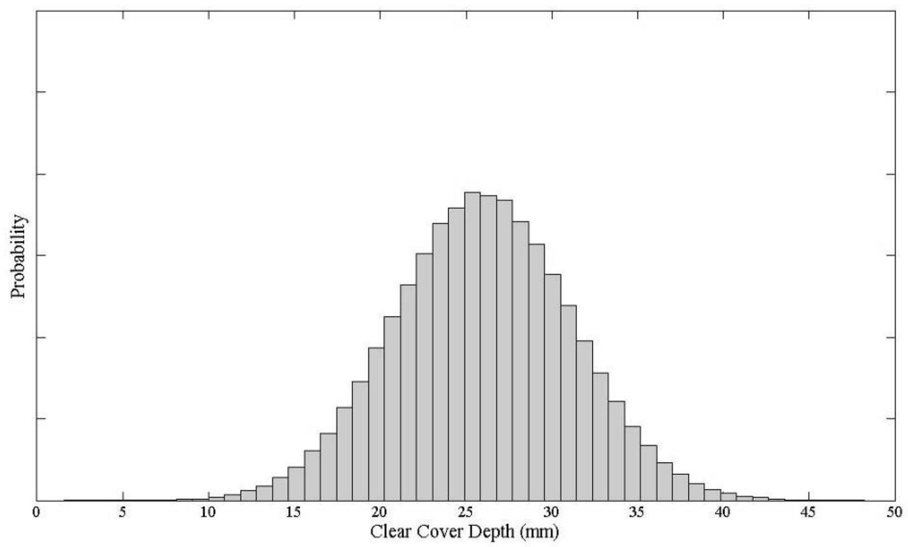
Appendix Figure E1 PDF of Chloride corrosion initiation concentration, $C_{(x,t)}$



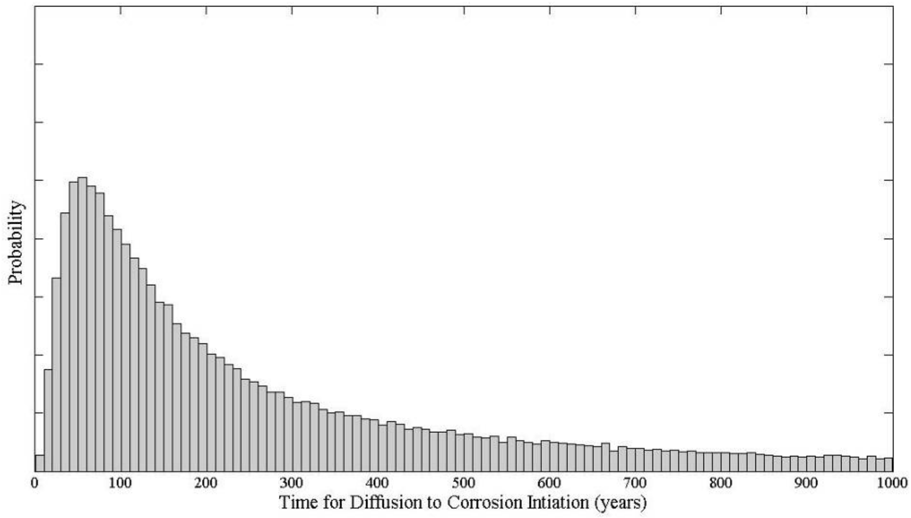
Appendix Figure E2 PDF of Surface chloride concentration, C_o (slab)



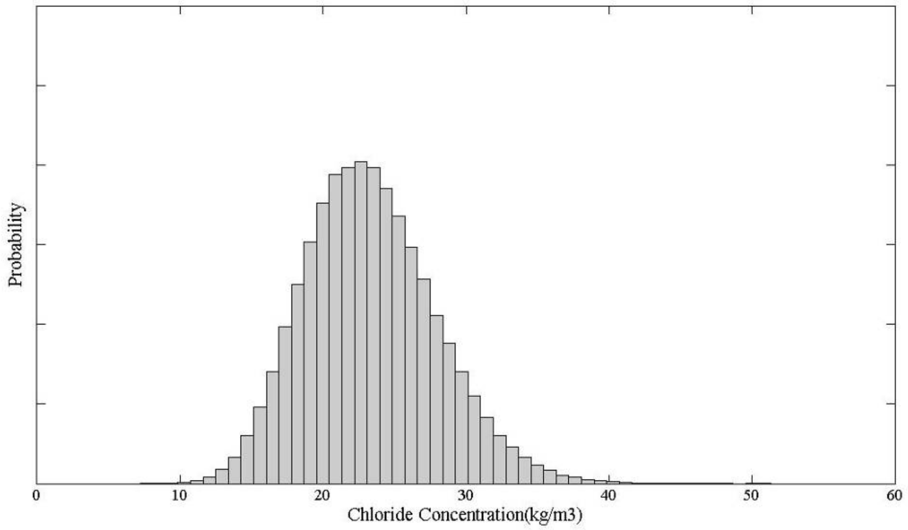
Appendix Figure E3 PDF of Apparent diffusion coefficient, D_c (slab)



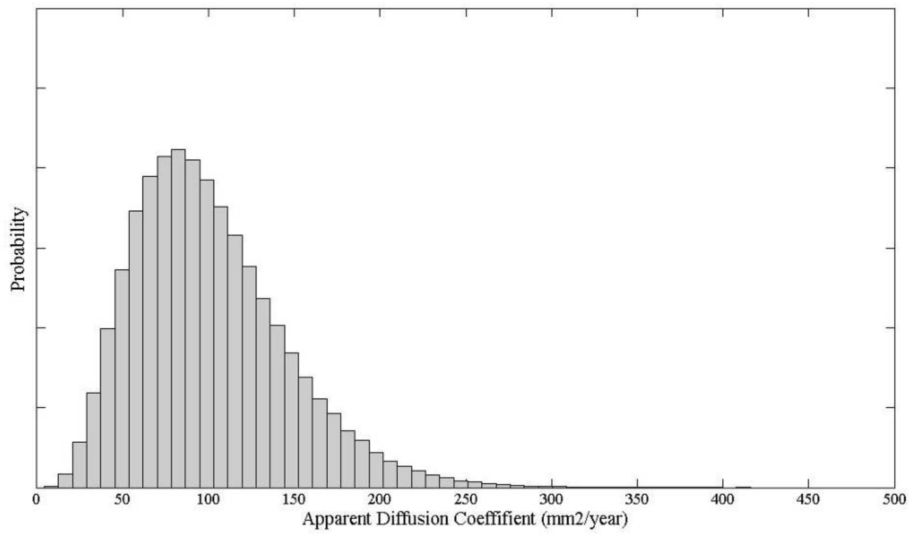
Appendix Figure E4 PDF of Cover depth, *x* (slab)



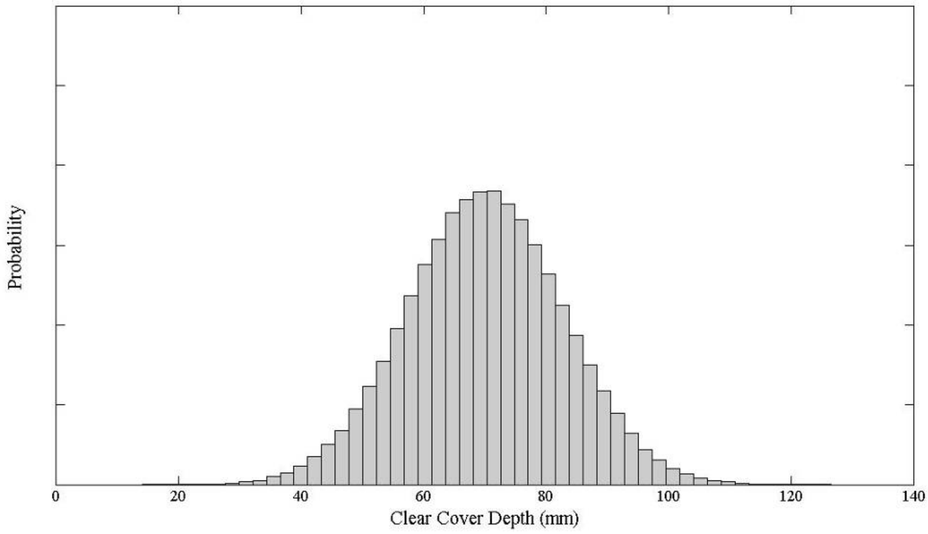
Appendix Figure E5 PDF of Time for diffusion to corrosion initiation (slab)



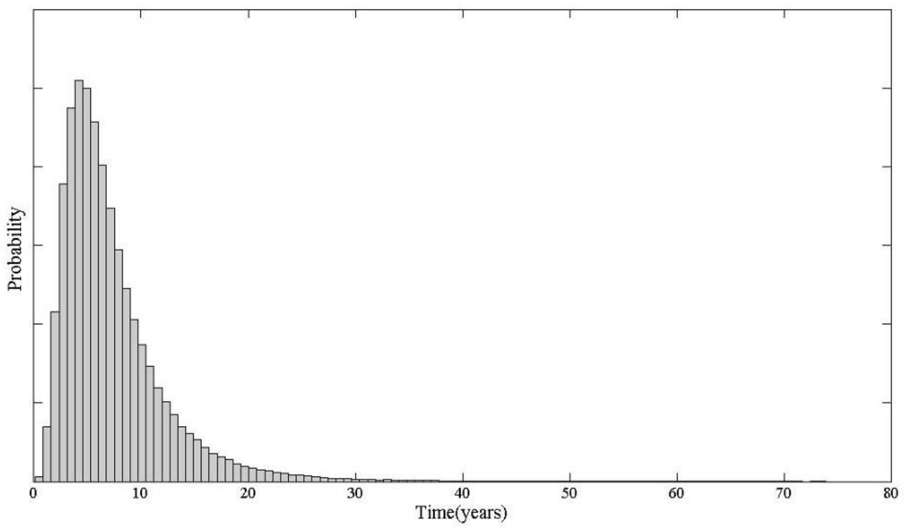
Appendix Figure E6 PDF of Surface chloride concentration, C_o (pier)



Appendix Figure E7 PDF of Apparent diffusion coefficient, D_c (pier)



Appendix Figure E8 PDF of Cover depth, *x* (pier)



Appendix Figure E9 PDF of Time for diffusion to corrosion initiation (pier)

Appendix F

Results for optimization

Appendix Table F1 Optimization solution for slab with corrosion rate 0.045 cm/year

Year	Alternative 1(Minor repair)			Alternative 2 (Major repair)			Alternative 3 (Varian repair)		
	Index	Action	Cost	Index	Action	Cost	Index	Action	Cost
0	4.573	-	-	4.573	-	-	4.573	-	-
2	4.573	-	-	4.573	-	-	4.573	-	-
4	4.573	-	-	4.573	-	-	4.573	-	-
6	4.573	-	-	4.573	-	-	4.573	-	-
8	4.573	-	-	4.573	-	-	4.573	-	-
10	4.573	-	-	4.573	-	-	4.573	-	-
12	4.573	-	-	4.573	-	-	4.573	-	-
14	4.573	-	-	4.573	-	-	4.573	-	-
16	4.573	-	-	4.573	-	-	4.573	-	-
18	4.573	-	-	4.573	-	-	4.573	-	-
20	4.573	-	-	4.573	-	-	4.573	-	-
22	4.573	-	-	4.573	-	-	4.573	-	-
24	4.573	-	-	4.573	-	-	4.573	-	-
26	4.573	-	-	4.573	-	-	4.573	-	-
28	4.573	-	-	4.573	-	-	4.573	-	-
30	4.419	-	-	4.419	-	-	4.419	-	-
32	4.264	-	-	4.264	-	-	4.264	-	-
34	4.110	-	-	4.110	-	-	4.110	-	-
36	3.956	-	-	3.956	-	-	3.956	-	-
38	3.801	-	-	3.801	-	-	3.801	-	-
40	3.647	-	-	3.647	-	-	3.647	-	-
42	3.493	-	-	3.493	-	-	3.493	-	-
44	3.338	-	-	3.338	-	-	3.338	-	-
46	3.184	-	-	3.184	-	-	3.184	-	-
48	3.030	-	-	3.030	-	-	3.030	-	-
50	2.875	-	-	2.875	-	-	2.875	-	-
52	2.721	-	-	2.721	-	-	2.721	-	-
54	2.567	-	-	2.567	-	-	2.567	-	-
56	2.412	-	-	2.412	-	-	2.412	-	-
58	2.258	-	-	2.258	-	-	2.258	-	-
60	2.10-2.83	Minor	4.55	2.10-4.10	Major	9.09	2.10-2.80	Minor	4.55
62	2.649	-	-	3.949	-	-	2.649	-	-
64	2.495	-	-	3.795	-	-	2.495	-	-
66	2.341	-	-	3.641	-	-	2.341	-	-
68	2.186	-	-	3.486	-	-	2.186	-	-
70	2.03-2.73	Minor	2.54	3.332	-	-	2.03-2.73	Minor	2.54
72	2.578	-	-	3.178	-	-	2.578	-	-
74	2.423	-	-	3.023	-	-	2.423	-	-
75	2.346	-	-	2.946	-	-	2.346	-	-
			7.08			9.09			7.08

Appendix Table F2 Optimization solution for slab with corrosion rate 0.055 cm/year

Year	Alternative 1(Minor repair)			Alternative 2 (Major repair)			Alternative 3 (Varian repair)		
	Index	Action	Cost	Index	Action	Cost	Index	Action	Cost
0	4.573	-	-	4.573	-	-	4.573	-	-
2	4.573	-	-	4.573	-	-	4.573	-	-
4	4.573	-	-	4.573	-	-	4.573	-	-
6	4.573	-	-	4.573	-	-	4.573	-	-
8	4.573	-	-	4.573	-	-	4.573	-	-
10	4.573	-	-	4.573	-	-	4.573	-	-
12	4.573	-	-	4.573	-	-	4.573	-	-
14	4.573	-	-	4.573	-	-	4.573	-	-
16	4.573	-	-	4.573	-	-	4.573	-	-
18	4.573	-	-	4.573	-	-	4.573	-	-
20	4.573	-	-	4.573	-	-	4.573	-	-
22	4.573	-	-	4.573	-	-	4.573	-	-
24	4.573	-	-	4.573	-	-	4.573	-	-
26	4.573	-	-	4.573	-	-	4.573	-	-
28	4.573	-	-	4.573	-	-	4.573	-	-
30	4.386	-	-	4.386	-	-	4.386	-	-
32	4.198	-	-	4.198	-	-	4.198	-	-
34	4.011	-	-	4.011	-	-	4.011	-	-
36	3.824	-	-	3.824	-	-	3.824	-	-
38	3.636	-	-	3.636	-	-	3.636	-	-
40	3.449	-	-	3.449	-	-	3.449	-	-
42	3.262	-	-	3.262	-	-	3.262	-	-
44	3.074	-	-	3.074	-	-	3.074	-	-
46	2.887	-	-	2.887	-	-	2.887	-	-
48	2.699	-	-	2.699	-	-	2.699	-	-
50	2.512	-	-	2.512	-	-	2.512	-	-
52	2.325	-	-	2.325	-	-	2.325	-	-
54	2.13-2.83	Minor	6.45	2.13-4.13	Major	12.90	2.13-4.13	Major	12.90
56	2.650	-	-	3.950	-	-	3.950	-	-
58	2.463	-	-	3.763	-	-	3.763	-	-
60	2.275	-	-	3.575	-	-	3.575	-	-
62	2.08-2.78	Minor	4.05	3.388	-	-	3.388	-	-
64	2.601	-	-	3.201	-	-	3.201	-	-
66	2.413	-	-	3.013	-	-	3.013	-	-
68	2.226	-	-	2.826	-	-	2.826	-	-
70	2.03-2.73	Minor	2.54	2.638	-	-	2.638	-	-
72	2.551	-	-	2.451	-	-	2.451	-	-
74	2.364	-	-	2.264	-	-	2.264	-	-
75	2.270	-	-	2.170	-	-	2.170	-	-
			13.04			12.90			12.90

Appendix Table F3 Optimization solution for slab with corrosion rate 0.065 cm/year

Year	Alternative 1(Minor repair)			Alternative 2 (Major repair)			Alternative 3 (Varian repair)		
	Index	Action	Cost	Index	Action	Cost	Index	Action	Cost
0	4.573	-	-	4.573	-	-	4.573	-	-
2	4.573	-	-	4.573	-	-	4.573	-	-
4	4.573	-	-	4.573	-	-	4.573	-	-
6	4.573	-	-	4.573	-	-	4.573	-	-
8	4.573	-	-	4.573	-	-	4.573	-	-
10	4.573	-	-	4.573	-	-	4.573	-	-
12	4.573	-	-	4.573	-	-	4.573	-	-
14	4.573	-	-	4.573	-	-	4.573	-	-
16	4.573	-	-	4.573	-	-	4.573	-	-
18	4.573	-	-	4.573	-	-	4.573	-	-
20	4.573	-	-	4.573	-	-	4.573	-	-
22	4.573	-	-	4.573	-	-	4.573	-	-
24	4.573	-	-	4.573	-	-	4.573	-	-
26	4.573	-	-	4.573	-	-	4.573	-	-
28	4.573	-	-	4.573	-	-	4.573	-	-
30	4.366	-	-	4.366	-	-	4.365	-	-
32	4.159	-	-	4.159	-	-	4.157	-	-
34	3.952	-	-	3.952	-	-	3.949	-	-
36	3.745	-	-	3.745	-	-	3.741	-	-
38	3.537	-	-	3.537	-	-	3.533	-	-
40	3.330	-	-	3.330	-	-	3.325	-	-
42	3.123	-	-	3.123	-	-	3.117	-	-
44	2.916	-	-	2.916	-	-	2.909	-	-
46	2.709	-	-	2.709	-	-	2.701	-	-
48	2.502	-	-	2.502	-	-	2.493	-	-
50	2.295	-	-	2.295	-	-	2.285	-	-
52	2.08-2.78	Minor	7.25	2.08-4.08	Major	14.49	2.08-4.08	Major	14.49
54	2.580	-	-	3.880	-	-	3.880	-	-
56	2.373	-	-	3.673	-	-	3.673	-	-
58	2.16-2.86	Minor	5.11	3.466	-	-	3.466	-	-
60	2.659	-	-	3.259	-	-	3.259	-	-
62	2.452	-	-	3.052	-	-	3.052	-	-
64	2.245	-	-	2.845	-	-	2.845	-	-
66	2.03-2.73	Minor	3.21	2.638	-	-	2.638	-	-
68	2.531	-	-	2.431	-	-	2.431	-	-
70	2.324	-	-	2.224	-	-	2.224	-	-
72	2.11-2.81	Minor	2.26	2.01-4.01	Major	4.52	2.01-2.71	Minor	2.26
74	2.609	-	-	3.809	-	-	2.509	-	-
75	2.506	-	-	3.706	-	-	2.612	-	-
			17.82			19.01			16.75

Appendix Table F4 Optimization solution for slab with corrosion rate 0.075 cm/year

Year	Alternative 1(Minor repair)			Alternative 2 (Major repair)			Alternative 3 (Varian repair)		
	Index	Action	Cost	Index	Action	Cost	Index	Action	Cost
0	4.573	-	-	4.573	-	-	4.573	-	-
2	4.573	-	-	4.573	-	-	4.573	-	-
4	4.573	-	-	4.573	-	-	4.573	-	-
6	4.573	-	-	4.573	-	-	4.573	-	-
8	4.573	-	-	4.573	-	-	4.573	-	-
10	4.573	-	-	4.573	-	-	4.573	-	-
12	4.573	-	-	4.573	-	-	4.573	-	-
14	4.573	-	-	4.573	-	-	4.573	-	-
16	4.573	-	-	4.573	-	-	4.573	-	-
18	4.573	-	-	4.573	-	-	4.573	-	-
20	4.573	-	-	4.573	-	-	4.573	-	-
22	4.573	-	-	4.573	-	-	4.573	-	-
24	4.573	-	-	4.573	-	-	4.573	-	-
26	4.573	-	-	4.573	-	-	4.573	-	-
28	4.573	-	-	4.573	-	-	4.573	-	-
30	4.336	-	-	4.336	-	-	4.336	-	-
32	4.100	-	-	4.100	-	-	4.100	-	-
34	3.863	-	-	3.863	-	-	3.863	-	-
36	3.626	-	-	3.626	-	-	3.626	-	-
38	3.389	-	-	3.389	-	-	3.389	-	-
40	3.153	-	-	3.153	-	-	3.153	-	-
42	2.916	-	-	2.916	-	-	2.916	-	-
44	2.679	-	-	2.679	-	-	2.679	-	-
46	2.442	-	-	2.442	-	-	2.442	-	-
48	2.20-2.90	Minor	9.15	2.20-4.20	Major	18.30	2.20-4.20	Major	18.30
50	2.669	-	-	3.969	-	-	3.969	-	-
52	2.432	-	-	3.732	-	-	3.732	-	-
54	2.19-2.89	Minor	6.45	3.495	-	-	3.495	-	-
56	2.659	-	-	3.259	-	-	3.259	-	-
58	2.422	-	-	3.022	-	-	3.022	-	-
60	2.18-2.88	Minor	4.55	2.785	-	-	2.785	-	-
62	2.648	-	-	2.548	-	-	2.548	-	-
64	2.412	-	-	2.312	-	-	2.312	-	-
66	2.17-2.87	Minor	3.21	2.07-4.07	Major	6.41	2.07-2.77	Minor	3.21
68	2.638	-	-	3.838	-	-	2.538	-	-
70	2.402	-	-	3.602	-	-	2.302	-	-
72	2.16-2.86	Minor	2.26	3.365	-	-	2.06-2.76	Minor	2.26
74	2.628	-	-	3.128	-	-	2.528	-	-
75	2.510	-	-	3.010	-	-	2.410	-	-
			25.6			24.71			23.77

Appendix Table F5 Optimization solution for slab with corrosion rate 0.085 cm/year

Year	Alternative 1(Minor repair)			Alternative 2 (Major repair)			Alternative 3 (Varian repair)		
	Index	Action	Cost	Index	Action	Cost	Index	Action	Cost
0	4.573	-	-	4.573	-	-	4.573	-	-
2	4.573	-	-	4.573	-	-	4.573	-	-
4	4.573	-	-	4.573	-	-	4.573	-	-
6	4.573	-	-	4.573	-	-	4.573	-	-
8	4.573	-	-	4.573	-	-	4.573	-	-
10	4.573	-	-	4.573	-	-	4.573	-	-
12	4.573	-	-	4.573	-	-	4.573	-	-
14	4.573	-	-	4.573	-	-	4.573	-	-
16	4.573	-	-	4.573	-	-	4.573	-	-
18	4.573	-	-	4.573	-	-	4.573	-	-
20	4.573	-	-	4.573	-	-	4.573	-	-
22	4.573	-	-	4.573	-	-	4.573	-	-
24	4.573	-	-	4.573	-	-	4.573	-	-
26	4.573	-	-	4.573	-	-	4.573	-	-
28	4.573	-	-	4.573	-	-	4.573	-	-
30	4.319	-	-	4.319	-	-	4.319	-	-
32	4.064	-	-	4.064	-	-	4.065	-	-
34	3.810	-	-	3.810	-	-	3.811	-	-
36	3.555	-	-	3.555	-	-	3.557	-	-
38	3.301	-	-	3.301	-	-	3.303	-	-
40	3.046	-	-	3.046	-	-	3.049	-	-
42	2.792	-	-	2.792	-	-	2.795	-	-
44	2.537	-	-	2.537	-	-	2.541	-	-
46	2.283	-	-	2.283	-	-	2.287	-	-
48	2.02-2.72	Minor	9.15	2.02-4.02	Major	18.30	2.02-4.02	Major	18.30
50	2.474	-	-	3.774	-	-	3.779	-	-
52	2.21-2.91	Minor	7.25	3.519	-	-	3.525	-	-
54	2.665	-	-	3.265	-	-	3.271	-	-
56	2.410	-	-	3.010	-	-	3.017	-	-
58	2.15-2.85	Minor	5.11	2.756	-	-	2.763	-	-
60	2.601	-	-	2.501	-	-	2.509	-	-
62	2.347	-	-	2.24-4.24	Major	8.09	2.255	-	-
64	2.09-2.79	Minor	3.60	3.992	-	-	2.00-2.70	Minor	3.60
66	2.538	-	-	3.738	-	-	2.447	-	-
68	2.283	-	-	3.483	-	-	2.19-2.89	Minor	2.85
70	2.02-2.79	Minor	2.54	3.229	-	-	2.639	-	-
72	2.474	-	-	2.974	-	-	2.385	-	-
74	2.220	-	-	2.720	-	-	2.131	-	-
75	2.093	-	-	2.593	-	-	2.004	-	-
			27.65			26.39			24.75

Appendix Table F6 Optimization solution for pier with corrosion rate 0.07 to 0.11cm/year

Year	Corrosion rate 0.07 cm/yr.			Corrosion rate 0.09 cm/yr.			Corrosion rate 0.11 cm/yr.		
	Index	Action	Cost	Index	Action	Cost	Index	Action	Cost
0	7.028	-	-	7.028	-	-	7.028	-	-
1	7.028	-	-	7.028	-	-	7.028	-	-
2	7.028	-	-	7.028	-	-	7.028	-	-
3	6.799	-	-	6.740	-	-	6.672	-	-
4	6.570	-	-	6.452	-	-	6.315	-	-
5	6.342	-	-	6.163	-	-	5.959	-	-
6	6.113	-	-	5.875	-	-	5.603	-	-
7	5.884	-	-	5.587	-	-	5.247	-	-
8	5.656	-	-	5.299	-	-	4.891	-	-
9	5.427	-	-	5.011	-	-	4.535	-	-
10	5.198	-	-	4.723	-	-	4.178	-	-
11	4.969	-	-	4.435	-	-	3.822	-	-
12	4.741	-	-	4.147	-	-	3.466	-	-
13	4.512	-	-	3.858	-	-	3.110	-	-
14	4.283	-	-	3.570	-	-	2.754	-	-
15	4.055	-	-	3.282	-	-	2.397	-	-
16	3.826	-	-	2.994	-	-	2.04-7.02	Major	39.36
17	3.597	-	-	2.706	-	-	6.672	-	-
18	3.368	-	-	2.418	-	-	6.315	-	-
19	3.140	-	-	2.13-7.28	Major	33.051	5.959	-	-
20	2.911	-	-	6.740	-	-	5.603	-	-
21	2.682	-	-	6.452	-	-	5.247	-	-
22	2.454	-	-	6.163	-	-	4.891	-	-
23	2.225	-	-	5.875	-	-	4.535	-	-
24	2.00-7.28	Major	24.698	5.587	-	-	4.178	-	-
25	6.799	-	-	5.299	-	-	3.822	-	-
26	6.570	-	-	5.011	-	-	3.466	-	-
27	6.342	-	-	4.723	-	-	3.110	-	-
28	6.113	-	-	4.435	-	-	2.754	-	-
29	5.884	-	-	4.147	-	-	2.397	-	-
30	5.656	-	-	3.858	-	-	2.04-7.02	Major	17.41
31	5.427	-	-	3.570	-	-	6.703	-	-
32	5.198	-	-	3.282	-	-	6.347	-	-
33	4.969	-	-	2.994	-	-	5.991	-	-
34	4.741	-	-	2.706	-	-	5.635	-	-
35	4.512	-	-	2.418	-	-	5.278	-	-
36	4.283	-	-	2.13-7.28	Major	12.274	4.922	-	-
37	4.055	-	-	6.740	-	-	4.566	-	-
38	3.826	-	-	6.452	-	-	4.210	-	-
39	3.597	-	-	6.163	-	-	3.854	-	-
40	3.368	-	-	5.875	-	-	3.497	-	-

Appendix Table F6 (Continued)

Year	Corrosion rate 0.07 cm/yr.			Corrosion rate 0.09 cm/yr.			Corrosion rate 0.11 cm/yr.		
	Index	Action	Cost	Index	Action	Cost	Index	Action	Cost
41	3.140	-	-	5.587	-	-	3.141	-	-
42	2.911	-	-	5.299	-	-	2.785	-	-
43	2.682	-	-	5.011	-	-	2.429	-	-
44	2.454	-	-	4.723	-	-	2.04-7.02	Major	7.701
45	2.225	-	-	4.435	-	-	6.672	-	-
46	2.00-7.28	Major	6.854	4.147	-	-	6.315	-	-
47	6.799	-	-	3.858	-	-	5.959	-	-
48	6.570	-	-	3.570	-	-	5.603	-	-
49	6.342	-	-	3.282	-	-	5.247	-	-
50	6.113	-	-	2.994	-	-	4.891	-	-
51	5.884	-	-	2.706	-	-	4.535	-	-
52	5.656	-	-	2.418	-	-	4.178	-	-
53	5.427	-	-	2.130-7.280	Major	4.558	3.822	-	-
54	5.198	-	-	6.740	-	-	3.466	-	-
55	4.969	-	-	6.452	-	-	3.110	-	-
56	4.741	-	-	6.163	-	-	2.754	-	-
57	4.512	-	-	5.875	-	-	2.397	-	-
58	4.283	-	-	5.587	-	-	2.04-7.02	Major	3.406
59	4.055	-	-	5.299	-	-	6.672	-	-
60	3.826	-	-	5.011	-	-	6.315	-	-
61	3.597	-	-	4.723	-	-	5.959	-	-
62	3.368	-	-	4.435	-	-	5.603	-	-
63	3.140	-	-	4.147	-	-	5.247	-	-
64	2.911	-	-	3.858	-	-	4.891	-	-
65	2.682	-	-	3.570	-	-	4.535	-	-
66	2.454	-	-	3.282	-	-	4.178	-	-
67	2.225	-	-	2.994	-	-	3.822	-	-
68	2.00-7.28	Major	1.902	2.706	-	-	3.466	-	-
69	6.799	-	-	2.418	-	-	3.110	-	-
70	6.570	-	-	2.13-7.28	Major	1.693	2.754	-	-
71	6.342	-	-	6.740	-	-	2.397	-	-
72	6.113	-	-	6.452	-	-	2.04-7.02	Major	1.507
73	5.884	-	-	6.163	-	-	6.672	-	-
74	5.656	-	-	5.875	-	-	6.315	-	-
75	5.427	-	-	5.587	-	-	5.959	-	-
			33.454			51.576			69.389

Appendix Table F7 Optimization solution for pier with corrosion rate 0.13 to 0.15 cm/year

Year	Corrosion rate 0.13 cm/yr.			Corrosion rate 0.15 cm/yr.		
	Index	Action	Cost	Index	Action	Cost
0	7.028	-	-	7.028	-	-
1	7.028	-	-	7.028	-	-
2	7.028	-	-	7.028	-	-
3	6.607	-	-	6.553	-	-
4	6.187	-	-	6.078	-	-
5	5.767	-	-	5.602	-	-
6	5.346	-	-	5.127	-	-
7	4.926	-	-	4.652	-	-
8	4.505	-	-	4.177	-	-
9	4.085	-	-	3.702	-	-
10	3.665	-	-	3.227	-	-
11	3.244	-	-	2.752	-	-
12	2.824	-	-	2.278-7.208	Major	49.697
13	2.403-7.280	Major	46.884	6.553	-	-
14	6.607	-	-	6.078	-	-
15	6.187	-	-	5.602	-	-
16	5.767	-	-	5.127	-	-
17	5.346	-	-	4.652	-	-
18	4.926	-	-	4.177	-	-
19	4.505	-	-	3.702	-	-
20	4.085	-	-	3.227	-	-
21	3.665	-	-	2.752	-	-
22	3.244	-	-	2.277	-	-
23	2.824	-	-	2.278-7.208	Major	26.180
24	2.403-7.280	Major	24.698	6.078	-	-
25	6.607	-	-	5.602	-	-
26	6.187	-	-	5.127	-	-
27	5.767	-	-	4.652	-	-
28	5.346	-	-	4.177	-	-
29	4.926	-	-	3.702	-	-
30	4.505	-	-	3.227	-	-
31	4.085	-	-	2.752	-	-
32	3.665	-	-	2.277	-	-
33	3.244	-	-	2.278-7.208	Major	14.619
34	2.824	-	-	6.078	-	-
35	2.403-7.280	Major	13.011	5.602	-	-
36	6.607	-	-	5.127	-	-
37	6.187	-	-	4.652	-	-
38	5.767	-	-	4.177	-	-
39	5.346	-	-	3.702	-	-
40	4.926	-	-	3.227	-	-
41	4.505	-	-	2.752	-	-
42	4.085	-	-	2.277	-	-

Appendix Table F7 (Continued)

Year	Corrosion rate 0.13 cm/yr.			Corrosion rate 0.15 cm/yr.		
	Index	Action	Cost	Index	Action	Cost
43	3.665	-	-	2.278-7.208	Major	8.163
44	3.244	-	-	6.078	-	-
45	2.824	-	-	5.602	-	-
46	2.403-7.280	Major	6.854	5.127	-	-
47	6.607	-	-	4.652	-	-
48	6.187	-	-	4.177	-	-
49	5.767	-	-	3.702	-	-
50	5.346	-	-	3.227	-	-
51	4.926	-	-	2.752	-	-
52	4.505	-	-	2.277	-	-
53	4.085	-	-	2.278-7.208	Major	4.558
54	3.665	-	-	6.078	-	-
55	3.244	-	-	5.602	-	-
56	2.824	-	-	5.127	-	-
57	2.403-7.280	Major	3.610	4.652	-	-
58	6.607	-	-	4.177	-	-
59	6.187	-	-	3.702	-	-
60	5.767	-	-	3.227	-	-
61	5.346	-	-	2.752	-	-
62	4.926	-	-	2.277	-	-
63	4.505	-	-	2.278-7.208	Major	2.545
64	4.085	-	-	6.078	-	-
65	3.665	-	-	5.602	-	-
66	3.244	-	-	5.127	-	-
67	2.824	-	-	4.652	-	-
68	2.403-7.280	Major	1.902	4.177	-	-
69	6.607	-	-	3.702	-	-
70	6.187	-	-	3.227	-	-
71	5.767	-	-	2.752	-	-
72	5.346	-	-	2.277	-	-
73	4.926	-	-	2.278-7.208	Major	1.421
74	4.505	-	-	6.078	-	-
75	4.085	-	-	5.602	-	-
			96.959			107.183

CIRRICULUM VITAE

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