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Probabilistic Deterioration Prediction of Prestressed Concrete Bridge Girder in a Chloride Environment

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Abstract

In recent years, aging of infrastructure has become a serious concern in Thailand. The performance of structure shows signs of deterioration that need for maintenance in order to ensure safety and serviceability of structure. Especially, prestressed concrete bridge girder that have widely used in Thailand and its need specific techniques for repairing. Bridge Management Systems (BMSs) have been developed to manage the maintenance of the bridges under required performance of structure and limitation of budget. Consequently, the purpose of this paper was to study deterioration prediction of prestressed concrete bridge girder that is one part of Bridge Management Systems (BMSs). Deterioration prediction model of performance of prestressed concrete is conducted in time dependent analysis. Due to corrosion of reinforcing steel by chloride attack is more severe than others, this study intends to introduce BMSs for chloride attack firstly. Criteria can be classified into three limit states, durability, serviceability and ultimate limit states. A reliability of structure was computed based on Monte Carlo simulation. The results demonstrate effect of parameters such as surface chloride content, chloride diffusion, corrosion rate and covering depth of concrete on performance of structure. Furthermore, deterioration prediction of prestressed concrete bridge girder can be used for the maintenance planning to reduce maintenance cost and extend the service life of structure.

Keywords: Bridge Management Systems, Prestressed concrete, Deterioration Prediction, Monte Carlo Simulation, Maintenance planning

1. INTRODUCTION

In recent years, deterioration of infrastructure has become a serious problem in Thailand. In order to ensure safety and serviceability of structures, maintenance of the structures is required. Additionally, Thailand is still progressing new infrastructure construction to carry on its economic capability with the world, which means Thailand may have to manage both of aging structure and new construction at the same time. Especially, prestressed concrete girder which is a major component part of road and elevated bridges structure in Thailand. As the prestressed concrete girders have become aging structure, maintenance of this structural is required and techniques are different form reinforced concrete. It have been reported that the prestressed concrete girders is possible but repair of unique method is required [1].

Deterioration of prestressed concrete can be occurred due to several causes. For example, impact by vehicle, environmental distress, extreme events etc. One of a global problem for concrete structure is corrosion of metal in concrete structure due to chloride attack. In addition, Thailand has more than 2600 km of coast line that some part of prestressed concrete bridges or future projects are located close to the sea. So, structure will be deteriorated faster due to chloride attack. Sancharoen.P et al. [2] have been reported deterioration of bridges that located near the coastline. It was found that chloride attack is a major cause of damages observed on structure nearby the sea.

In case of prestressed concrete structure, prestressing strand is more serious than reinforcing steel, because failure can lead to collapse of structure suddenly. Also stress in prestrssing strand can exceed the limitation of allowable stress due to corrosion (about 55% to 65% of its ultimate tensile strength). It not only breakdown on yielding of prestressing steel, other types of deterioration mechanisms in prestressing steel may have occurred by high stress level of prestressing steel and corrosion in anchorage also[3].

Similarly, the Lowe's walkway bridge across to motor speedway in North Carolina which is prestressed concrete structure was collapsed due to corrosion of prestressing steel as shown in Fig 1. According to inspection, corrosion product around the prestessing strand in the bridge was a cause of bridge structure failure. Calcium chloride which cause a corrosion was founded in grout around prestressing steel. The disaster of bridge structure was caused by 11 prestressing steel in concrete were cut by corrosion. Also, because bridge is a private structure which was lacked of inspection to check the possibility of any damage from the government office. The sign of corrosion deterioration of bridge may have been observed, if the bridge had been carefully investigated [4].

Repair methods need to be considered for effectively utilizing the budget with the condition such as structural performance, safety to user, socials or commercials, environment and limited budget. Bridge Management Systems (BMSs) are intended to operate the maintenance of bridges under the limitation of budget and degradation of structural performance. Many agencies have been developed the BMSs products to support this maintenance planning well known the BRIDGIT and Pontis system in The United States of America. The BMSs has classified into three parts that comprise in assessing bridge condition, the deterioration prediction and decision to the maintenance of bridges [5, 6].

Figure 1 Bridge structure collapse due to corrosion failure [4]

The deterioration prediction is one of most important part of BMSs for maintenance planning, many degradation prediction have been evaluated by a score in visual investigation of structure soundness and mostly in reinforced concrete structure. Prestressed concrete bridges structure, popular in Thailand, a few studies have been developed in actual inspection results for evaluation and prediction of prestressed concrete bridges. Moreover, prestressed concrete behavior is different from reinforced concrete such as high stress level of materials and loss of prestress.

Investigation of structure to evaluate the performance of actual structure, accuracy of deterioration prediction can be improved [7]. So, probabilistic approach is appropriately improved in the deterioration prediction [8]. In addition, There are some of researches [9, 10] reported that the Monte Carlo simulation is more traditional probabilistic technique and very useful tool for engineers for estimating reliability or probability of failure of involved engineering systems. In order to ensure safety and serviceability of prestressed concrete girder, maintenance planning must be considered. Therefore, this study focuses on probabilistic deterioration prediction of prestressed concrete bridge girder due to chloride attack is a part of maintenance planning.

2. METHODOLOGY

There are three main stages of probability of failure consisting of input the parameters for simulation, computed the structural performance with different criteria and summarized the probability of failure as shown in Fig 2. Equations used in this study are mainly from design standards such as AASHTO, ACI, JSCE and DPT. Actual condition of structure are considered by inspected parameters to improve the accuracy of the model.

In this study, probability of failure of prestressed concrete bridge girder was considered according to three limit states durability, serviceability and ultimate limit states.

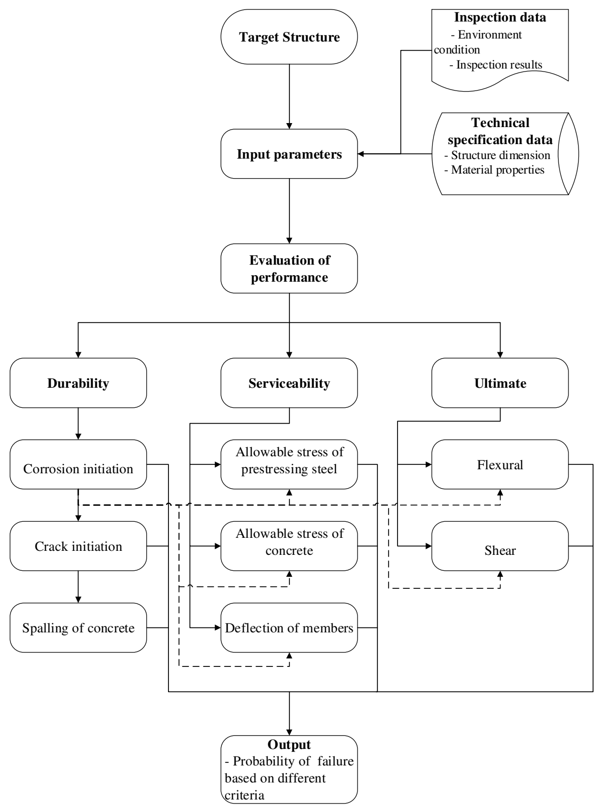

Figure 2 Conceptual framework

2.1 Performance criteria

2.1.1 Durability limit states

The durability criteria of concrete for service life of structure are depending on their environment. The processes of corrosion are classified to stages as corrosion initiation, corrosion inducing concrete to crack and spalling respectively.

- *Corrosion initiation*

The ingress of chloride ions (Cl^{-}) diffused into the surface concrete. Corrosion is initiated when Cl^{-} concentration exceeds critical chloride content (Cl_{lim}) as 0.4% by weight of binder as shown in Equation (1) [11].

Chloride content at steel surface could be calculated from Equation (2). [11]

$$Cl_d < Cl_{lim} \quad (1)$$

in which

$$Cl_d = \frac{Cl_s\left[1-\text{erf}\left(\frac{0.1C}{2\sqrt{D_a \times t_r}}\right)\right]}{B} \quad (2)$$

where Cl_d is chloride concentration (weight of binder), Cl_s is chloride concentration at surface of concrete (kg/m^3), C is covering depth (mm), B is weight of binder (kg/m^3), D_a is chloride Diffusion (cm^2/year) and t_r is remaining time (year).

- *Steel corrosion mass loss*

Many researches [12-15] have used Faraday's law to calculate the weight loss of steel from corrosion, and its ability to predict the actual loss of steel. It can be calculated using the following expression based on Faraday's law as shown in Equation (3).

$$M_{loss} = \frac{MIT}{ZF} \quad (3)$$

where T is the time (s), M_{loss} is the mass of steel lost in time (g) to form rust, I is the current (A), F is Faraday's constant (96,500 A s), z is the ionic charge (2 For Fe), and M is the atomic mass of the metal (56 g for Fe).
Hence, the relationship between the time, T, and the mass of steel consumed to form rust, M_{loss} is given by

$$T = \frac{24125\,(M_{loss}/a_s)}{7i} \quad (4)$$

where i is the current density (A/cm^2), and a_s is the surface area of the steel reinforcing bar. The ratio (M_{loss} /a_s) can be expressed as follows:

$$\frac{M_{loss}}{a_s} = \frac{(m_l)(\frac{\pi D^2}{4})\rho_s}{100(\pi D)} = \frac{m_l D\,\rho_s}{400} \quad (5)$$

where m_l is the percentage steel mass loss, and ρ_s is the mass density of steel (7.85 g/cm^3).

Combination Equation (4) and (5) to determine the relationship between the percentage steel mass loss, $m_l$and time, it has been summarized the formula that can be expressed as Equation (6). [13]

$$m_l = \frac{T\,i_{corr}}{78.3D} \quad (6)$$

where T is the time (days), m_l is the percentage of steel mass loss, D is diameter of the steel reinforcing bar (mm) and i_{corr} is the current density in ($\frac{\mu A}{cm^2}$).

- *Corrosion induced concrete cracking*

The time to first cracking, t_{cr} estimated using empirical model [16] which is relating to the amount of corrosion product, $W'_{steel,cr}$ and corrosion current density in steel, i_{corr} was estimated based on Faraday's law by using Equation (7).

$$t_{cr} = \frac{W'_{steel,cr}}{0.009113\,i_{corr}} \quad (7)$$

where, t_{cr} is time of corrosion induced concrete cracking in (year), $W'_{steel,cr}$ is the critical mass loss of steel that equal to 0.01 g/cm^2 [7] and i_{corr} is the current density in ($\frac{\mu A}{cm^2}$).

- *Corrosion induced spalling of concrete cover*

Cracks often propagate to the surface resulting in concrete spalling or loss of bond. Time to spalling of concrete cover (t_{sp}) determined by using empirical models [17, 18] as shown in Equation (8).

$$t_{sp} = [0.84\,(t_{ser} + t_{cr} + 0.2)]^{1.4} \quad (8)$$

where t_{sp} is time of corrosion induced concrete spalling in (year), t_{ser} is time to severe cracking in (year) and t_{cr} is time to initiation cracking from Equation (7).
in which

$$t_{ser} = [A \times 10^{-3} \times (wc/C)^{-B}] \times \frac{100}{i_{corr}} \quad (9)$$

where wc is water to cement ratio, C is covering depth (mm) and A is 6.5 and B is 0.57 for a limit crack width of 0.5mm

2.1.2 Serviceability limit states

Serviceability limit state shall be stress limitations for concrete and prestressing steel and allowable deflection.

- *Stress limitations for prestressing steel*

As a result of reduction in prestressing steel sectional area due to corrosion has effected to increasing of stress. Consequently, the allowable stress of steel form steel section loss shall not exceed AASHTO LRFD Bridge Design Specifications [19].

- *Stress limitations for concrete*

Allowable stress of concrete are expressed as functions of compressive strength at that time whereas and shall not exceed AASHTO LRFD Bridge Design Specifications [19].

- *Deflection*

The deflection of a flexural member is calculated to satisfy a limit state of serviceability. The total deflection is a resultant of the upward deflection due to prestressing force, downward deflection due to the gravity loads which including self-weight and live load.

Determining prestressed loss, affect directly to deflection of prestressing force is conforming to refined estimates of time-dependent losses of AASHTO LRFD Bridge Design Specifications [19]. Excepting of shrinkage strain is conformed to DPT 1332 which are suitable for environmental condition in Thailand [11]. The deflection limitation shall be considered for concrete construction which the general vehicle load limited to span/800 according to AASHTO LRFD Bridge Design Specifications [19].

2.1.3 Ultimate limit states

Both of flexural and shear capacity was calculated from section and material properties, and considering strength reduction factor. The nominal both of flexural and shear capacity with strength reduction factor shall exceed to ultimate moment and shear respectively which can be determined from structural analysis according to AASHTO LRFD Bridge Design Specifications [19].

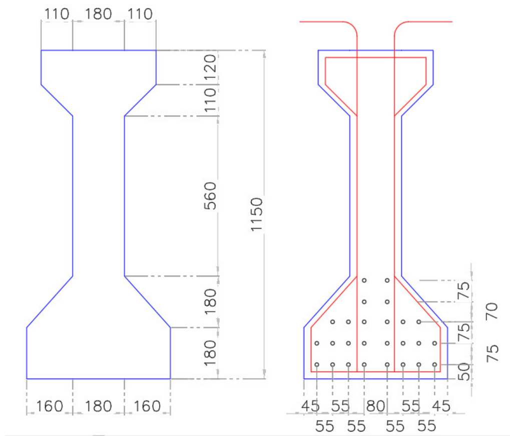

Figure 3 Dimension of prestressed concrete girder (mm.)

2.2 Monte Carlo simulation and case studies

Monte Carlo simulation can be expressed as a statistical analysis, which generating samples of random numbers in interval of [0, 1] are activated to perform the simulation. The process is simulated by use of three limit states for evaluating the performance of structure such as durability, serviceability and ultimate states. The simulation technique is used for computing the probability of failure on each criterion. The probability of failure (p_f) can be estimated by Equation (10).

$$p_f = \frac{1}{N} \cdot \sum_{j=1}^{N} I\left[g(X_j)\right] \quad (10)$$

where N is the number of simulation, $I\left[g(X_j)\right]$ is the indicator function and X_j is the *j*th sample drown according to the probability density function *f*x(x).

The prestressed concrete girder considered in this case study is a properly seven girders of a simple span prestressed concrete bridge in Bangkok (constructed in 1981) which has span length of 20 m, 0.18 m effective slab thickness, 1.975 m effective width and dimension of girder as shown in Fig 3. The girder was designed according to the AASHTO LRFD Bridge Design Specifications. The primary live load was calculated based on vehicle of Highway Load 1993 (HL-93).

The design concrete strength (f_c') is 40 MPa at 28 days. All prestressing strands are uncoated 7-wire stress relived strands (12.4 mm) grade 250 which conform to ASTM A416-68 and have the ultimate tensile strength of the prestressing strand is 1725 MPa. Clear cover of reinforcement is 25 mm. Reinforcing steel conforms to the requirement of AASHTO M31, Grade 40 that the diameter is 12 mm. The bridges design assumes to concern only dead load and live load. Table 1 show the parameters which can get from inspection and assumption. The analysis considers the variability and uncertainty of covering of concrete, yield strength of prestressing steel, chloride diffusion and corrosion rate.

Table 1 Parameters of prediction

Parameter	G1	G2	G3	G4	G5	G6	G7	Reference
Thickness of concrete cover (mm)	25	25	25	20	15	25	25	[20]
COV	0.16	0.16	0.16	0.16	0.16	0.16	0.16	
f_{py}, Yield strength (MPa)	1725	1725	1725	1725	1725	1725	1725	[20]
COV	0.28	0.28	0.28	0.28	0.28	0.28	0.28	
Surface chloride content (% of binder)	2.95	2.95	2.95	2.95	2.95	2.95	2.95	[21]
COV	0.1	0.1	0.1	0.1	0.1	0.1	0.1	
Chloride diffusion (cm^2/years)	0.3	0.4	0.5	0.3	0.3	0.3	0.3	[21]
COV	0.2	0.2	0.2	0.2	0.2	0.2	0.2	
i_{corr}, Corrosion rate ($\mu A/cm^2$)	1	2	3	1	1	1	1	[21]
COV	0.2	0.2	0.2	0.2	0.2	0.2	0.2	
Number of tendon	26	26	26	26	26	24	22	

3. RESULTS AND DISCUSSION

3.1 Probability of failure based on corrosion initiation, crack and spalling of concrete criteria

3.1.1 Effect of chloride diffusion and corrosion rate

Corrosion initiation, crack and spalling of concrete results can be seen in Fig 4 and 5 which show that aggressive chloride diffusion and corrosion rate tends to have higher probability of failure. For corrosion initiation, increase chloride diffusion from 0.3 to 0.5 (cm^2/years) can lead to reduction of service life from about 2.5 to 5 years. For crack and spalling of concrete are related to corrosion rate which increase corrosion rate from 1 to 3 ($\mu A/\mathrm{cm}^2$), it can lead to decrease the service life about 2.5 to 5 years for cracking but for spalling can lead to reduce the performance about 20 to 25 years.

3.1.2 Effect of concrete cover

Fig 6 and 7 show results of probability of failure due to durability. It shows that less of concrete cover tends to have higher probability of failure. For corrosion initiation, reduce covering of concrete from 25 to 15 (mm) results in increase of probability of failure sharply to 5 - 7.5 years. For crack and spalling of concrete have the same tendency on time dependent probability of failure of prestressed concrete bridge girder that probability of failure if the covering of concrete reduces. The results of probability of failure in cracking increase from about 2.5 to 5 years. Similarly results of spalling of concrete that rise to 15 - 20 years.

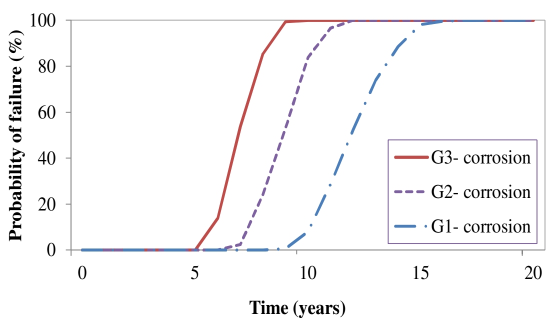

Figure 4 Effect of chloride diffusion on corrosion initiation criteria

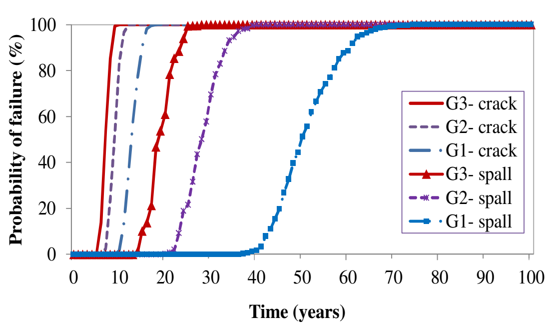

Figure 5 Effect of chloride diffusion and corrosion rate on crack and spalling of concrete criteria

3.2 Probability of failure based on allowable stress of concrete and prestressing steel criteria

3.2.1 Effect of chloride diffusion and corrosion rate

Fig 8 presented probability of failure due to serviceability limit states. It was found that G3 is severe damage in case of allowable stress in concrete because of violent of chloride diffusion as well as in case of allowable stress in prestressing steel. So, it can be conclude that aggressive environment tends to have higher probability of failure.

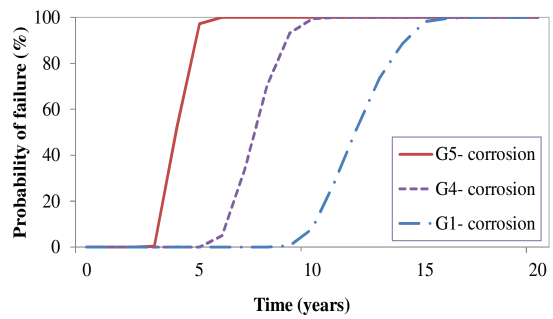

Figure 6 Effect of concrete cover on corrosion initiation criteria

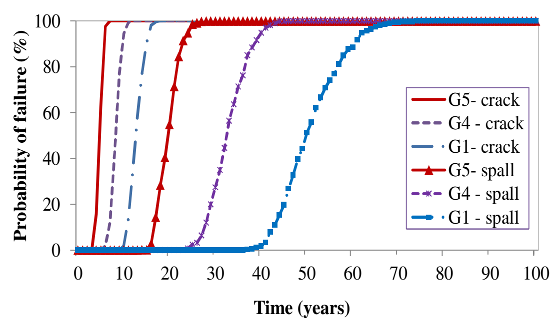

Figure 7 Effect of concrete cover on crack and spalling of concrete criteria

3.2.2 Effect of concrete cover

The results of probability of failure due to allowable stress of concrete and presressing steel are discussed in Fig 9. It can be seen that probability of failure, both of allowable stress in concrete and prestressing steels are increase to 10 years due to reduction of 5 mm of covering of concrete Especially, G5 is the most concerned because thickness of concrete cover is smaller than other.

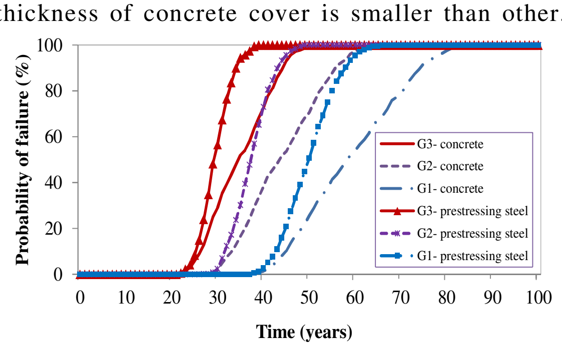

Figure 8 Effect of chloride diffusion and corrosion rate on allowable of concrete and prestressing tendon criteria

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3.3 Probability of failure based on flexural and shear strength criteria

3.3.1 Effect of chloride diffusion and corrosion rate

Fig 10 presented G3 is the most considered of both of flexural and shear strength because it was indicated that violent environment inclines to have higher probability of failure. However, shear strength is more serious than flexural strength because of different covering of concrete. In additional, probability of failure due to shear seems like severe damage than service states because corrosion rate in assumption is very strong for loss of steel section.

3.3.2 Effect of concrete cover

The results of probability of failure due to ultimate limit states are discussed in Fig 11. G5 is serious consideration due to covering of concrete are smallest both of flexural and shear strength. Moreover, it can be seen that time to failure, increasing of time to failure about 10 years for flexural strength and 5 years for shear strength from different concrete cover.

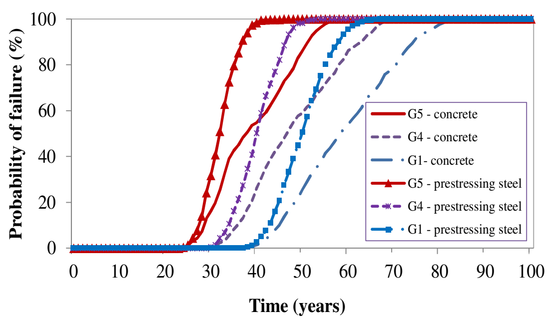

Figure 9 Effect of concrete cover on allowable of concrete and prestressing tendon criteria

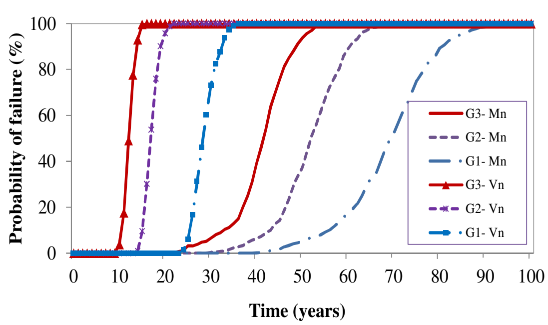

Figure 10 Effect of environment on flexural and shear strength criteria

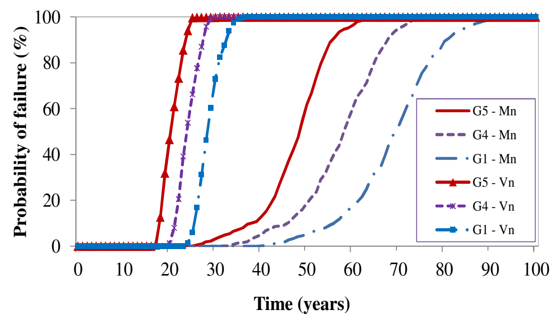

Figure 11 Effect of concrete cover on flexural and shear strength criteria

3.3.3 Effect of number of strand failure

Fig 12 presented the effect of different number of strand on time dependent probability of failure of prestressed concrete bridge girder, by comparing number of strand. Not surprisingly, this results show that number of strand is one of the primary variables that will directly affect the probability of failure for prestressing failure only. As can be seen the graph, the probability of failure reduce from 60 years to 50 years if the number of strands decrease by 2 strands.

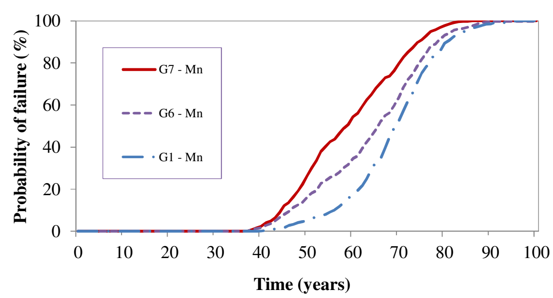

Figure 12 Effect of number of strand failure on flexural strength criteria

3.4 Priority of maintenance decision

Due to the different parameters of seven girders, there are different failure probabilities of these three criteria. The first, maintenance decision on durability limit states such as reinforcement corrosion, concrete cracking and spalling of concrete are considered. Fig 13 and 14 shows the priority of maintenance most to corrosion initiation and crack that G5 is the severe damage because the covering depth of prestressed concrete and the covering depth of stirrup are the smallest than other girders. The second severe girder is G3 because it has very high corrosion rate and chloride diffusion. In case of the ranking of failure due to spalling of concrete as shows in Fig 15, the results that G3 and G5 are the most concern because its failure increased very fast.

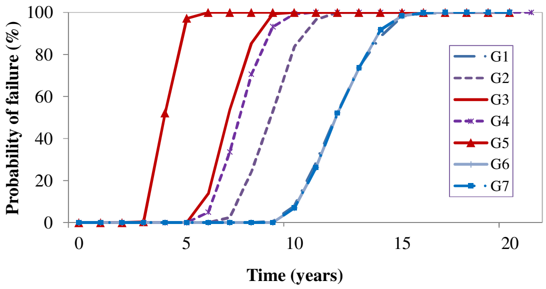

Figure 13 Ranking of failure due to corrosion initiation

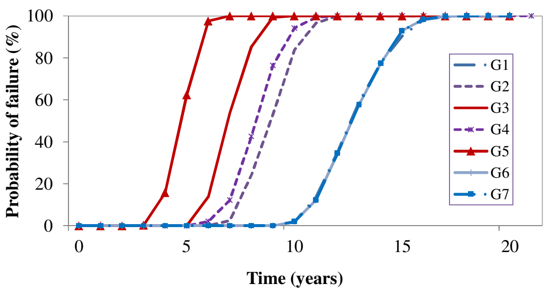

Figure 14 Ranking of failure due to crack initiation

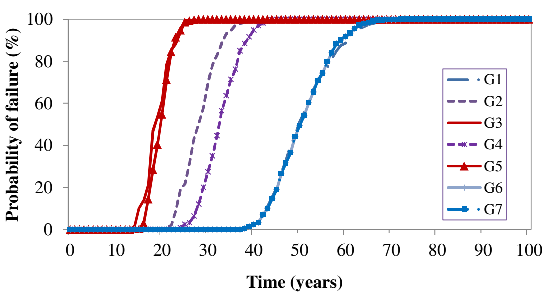

Figure 15 Ranking of failure due to spalling of concrete

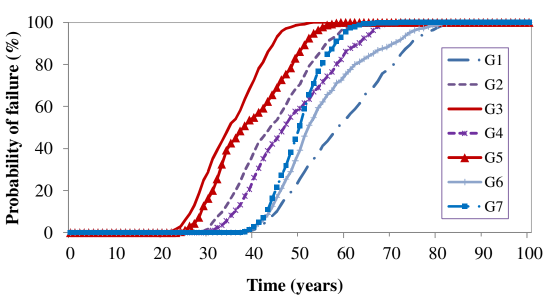

Figure 16 Ranking of failure due to allowable stress in concrete

The second, the tendency of failure due to serviceability limit states results on allowable stress of prestressing steel and concrete and indicated that the allowable stress of prestressing steel is more concerned than allowable stress of concrete as shown in Fig 16 and 17. G3 and G5 are still severe failure according to violets of corrosion and less of covering depth of concrete respectively.

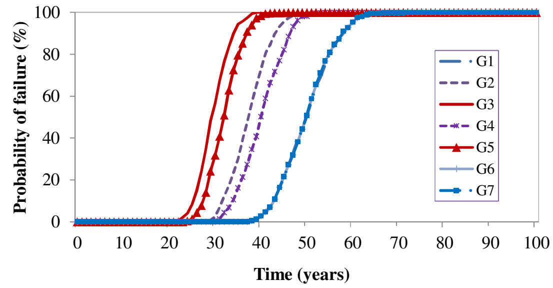

Figure 17 Ranking of failure due to allowable stress in prestressing tendons

Third, the priority of maintenance due to ultimate limit states had shown the arrangement of failure to shear strength and flexural moment strength. In case of shear strength the results shown that effect of environmental condition is the most severe damage followed by covering depth of concrete as shown in Fig 18. However, flexural strength criteria is indicated the tendency that effect of environmental condition is the most considered followed by covering depth of concrete and number of strands respectively as shown in Fig 19.

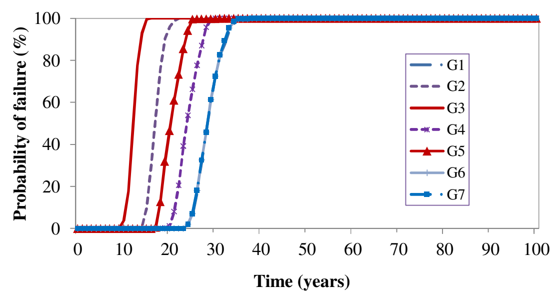

Figure 18 Ranking of failure due to shear strength

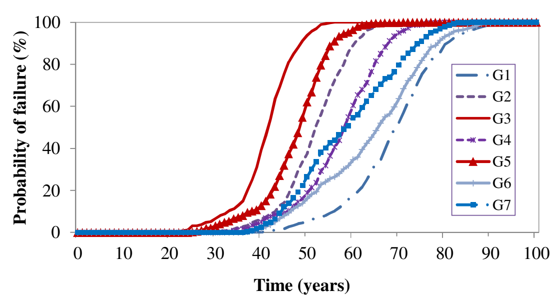

Figure 19 Ranking of failure due to flexural strength

4. CONCLUSIONS

The result of this study is performance of structure predicted based on the probability and variation of inspection results of prestressed concrete bridge girder. Mainly chloride attack is considered in this study, it can be concluded that:

- From the prediction, performance of prestressed concrete girder are significantly affected by chloride attack.

- The aggressive environment condition and covering depth of concrete show a significant effect on probability of failure for prestressed concrete bridge girder.
- Serviceability and safety of prestressed concrete girders are significantly affect by corrosion of steel. Large covering depth and low w/b concrete should be considered in the design stage.
- Based on predicted probability of failure, maintenance cost and expected failure cost can be calculated. Finally, maintenance planning and optimization can be conducted.

5. ACKNOWLEDGMENT

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6. REFERENCES

[1] L. Kasan. (2009). Structural repair of prestressed concrete bridge grider, master thesis, department of civil and environmental engineering, university of Pittsburgh, United State of America

[2] Sancharoen, P., Sahamitmongkol, R., Kato, Y., and Uomoto, T. (2007). Utilization of Inspection Result of RC Structure in Thailand to Plan Maintenance for Chloride Attack, JCI Annual Conference, Vol. 29, No.30, pp. 1687-1692

[3] ACI American Concrete Institute (2001). Corrosion of Prestressing steels. Detroit, MI

[4] GOINS. D. (2000). Motor speedway bridge collapse caused by corrosion, *Materials Performance*, 36 (7), 18-19.

[5] Ayaho Miyamoto et al. (2001). Development of bridge management system for existing bridges, Advances in engineering software. 3, 821-833.

[6] Hatem Elbehair. (2007). Bridge management system with integrated life cycle cost optimization, thesis, University of Waterloo, Ontario, Canada

[7] JSCE Japan Society of civil engineers. (2007). Standard specification for concrete structure Maintenance. Japan

[8] Sanjeev K V. et al. (2014). Probabilistic Evaluation of Service Life for Reinforced Concrete Structures. Chinese Journal of Engineering Volume 2014, Article ID 648438, 8 p

[9] S. Tesfamariam and B. Mart´ın-P´erez.(2008). Bayesian belief network to assess carbonation-induced corrosion in reinforced concrete. *Journal of Materials in Civil Engineering*, vol. 20, no. 11, pp. 707–71.

[10] Rui Miguel Ferreira. (2004). Probality based durability analysis of concrete structure in marine environment, Ph.D. dissertation. Department of Civil Engineering, School of Engineering, University of Minho,Portugal

[11] DPT Department of Public Works and Town & Country Planning, Ministry of Interior. (2007). Standard of concrete according to durability and service life. Thailand

[12] El Maaddawy T, Soudki K.(2003). Effectiveness of impressed current technique to simulate corrosion of steel reinforcement in concrete. ASCE J Mater Civ Eng;15 (1):41–7.

[13] El Maaddawy et al. (2007). A model for prediction of time from corrosion initiation to corrosion cracking. Cement & Concrete Composites 29 168–175

[14] Mohamed K. Et al. (2010). Detecting Corrosion of Steel Prestressing Strands Using Acoustic Emission. Annual Conference of the Prognostics and Health

[15] Chunhua Lu, Weiliang Jin, Ronggui Liu. (2011) Reinforcement corrosion-induced cover cracking and its time prediction for reinforced concrete structures, Corrosion Science 53 1337–1347

[16] Leon C. and Dimitri V. (2011). Prediction of corrosion-induced cover cracking in reinforced concrete structures. Construction and Building Materials 25, 1854-1869

[17] KAT Vu and MG Stewart. (2002). Service Life Prediction of Reinforced Concrete Structures Exposed to Aggressive Environments. 9DBMC- 2002 Paper 119, pp.1-10.

[18] P. Thoft-Christensen. (2010). Modeling of the Deterioration of Reinforced Concrete Structures. Proceedings IFIP WG 7.5 Conference on "Reliability and Optimization of Structural Systems", Ann Arbor, USA, September 25-27, pp. 15-26.

[19] AASHTO American Association of State Highway and Transportation Official (2007). Bridge Design Specification, AASHTO WASHINGTON D.C.

[20] Mojtaba Mahmoodian. (2012). Failure assessment of a pre-stressed concrete bridge using time dependent system reliability method29th International Bridge Conference, At Pittsburgh, PA, United State of America

[21] M. S. Darmawan and M. G. Stewart. (2007). Spatial time-dependent reliability analysis of corroding pretensioned prestressed concrete bridge girders, Structural Safety 29, 16-31.

7. BIOGRAPHIES

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