PROBABILISTIC DETERIORATION PREDICTION OF PRESSTRESSED CONCRETE BRIDGE GIRDER

BY

KRIT SUKPRASIT

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE (ENGINEERING AND TECHNOLOGY)
SIRINDHORN INTERNATIONAL INSTITUTE OF TECHNOLOGY
THAMMASAT UNIVERSITY
ACADEMIC YEAR 2016
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A Thesis Presented

By

KRIT SUKPRASIT

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AUGUST 2017
Abstract

PROBABILISTIC DETERIORATION PREDICTION OF PRESTRESSED CONCRETE BRIDGE GIRDER

by

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Bachelor of Engineering (Civil Engineering), Sirindhorn International Institute of Technology, Thammasat University, Thailand, 2013

Master of Science (Civil Engineering), Sirindhorn International Institute of Technology, Thammasat University, Thailand, 2017

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 thousands member on each route of expressway structure and elevated train structure need specific techniques for repairing. Bridge Management Systems (BMSs) have been developed to manage the maintenance of the bridges under the performance of structure and limitation of budget. Consequently, the purpose of this study was to create the software about deterioration prediction of prestressed concrete bridge girder that one part of Bridge Management Systems (BMSs).

Deterioration prediction model is proposed in primarily for providing the performance of prestressed concrete in time dependent analysis. In this study, criteria can be classified the criteria into three limit states that comprise with durability, serviceability and ultimate limit states for considering. A reliability of structure was computed based on Monte Carlo simulation. The results demonstrate to influences of performance by parameters such as carbonation concentration, chloride concentration,
chloride diffusion, corrosion rate and covering depth of concrete. Additionally, the probability of failure can be improved the accuracy of prediction and easily understood the performance of their criteria by non-mathematicians. In order to accumulate their information for displaying on the software, it used Microsoft access to manage the data and applied for ranking of maintenance decisions.

In summary, the uncertainties parameters that identify the selected computational models and their impacts can be evaluated by Monte Carlo simulation to improve the accuracy of prediction. Furthermore, this deterioration prediction of prestressed concrete bridge girder software can be supporting the maintenance management for planning to reduce maintenance cost and extend the service life of structure.

**Keywords**: Bridge Management Systems, Prestressed concrete, Deterioration Prediction, Monte Carlo Simulation
Acknowledgements

I would like to express my sincere gratitude to my supervisor, Dr. Pakawat Sancharoen for continuous supported of my master study and research. I am grateful for his guidance and knowledge during this research work. Beside I would like to express my deep gratitude to Prof. Dr. Somnuk Tangtermsirikul, the director of Sirindhorn International Institute of Technology (SIIT) and head of the Construction and Maintenance Technology Research Center (CONTEC) who gave me the invaluable opportunity to study in master. His discussion and immense knowledge make a great guidance throughout my studies.

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Furthermore, I would like to appreciate my parent, Mr.Supakorn and Mrs. Supapong Sukprasit, for all their support during this study.

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<td>$A_d$</td>
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<td>$A_g$</td>
<td>gross area, non-composite girder ($\text{mm}^2$)</td>
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<td>$A_{ps}$</td>
<td>area of prestressing steel ($\text{mm}^2$)</td>
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<td>$A_s$</td>
<td>area of nonprestressed tension reinforcement ($\text{mm}^2$)</td>
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<td>$A_v$</td>
<td>area of shear reinforcement within a distance ($\text{mm}^2$)</td>
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<td>$A_s'$</td>
<td>area of compression reinforcement ($\text{mm}^2$)</td>
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<td>$a$</td>
<td>depth of the equivalent stress block (mm)</td>
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<td>$B$</td>
<td>weight of binder ($\text{kg/m}^3$)</td>
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<td>$b$</td>
<td>width of compression face of the member (mm)</td>
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<td>$b_w$</td>
<td>web width of section (mm)</td>
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<tr>
<td>$C$</td>
<td>covering depth (mm)</td>
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<tr>
<td>$C_{ld}$</td>
<td>chloride concentration (percent by weight of binder)</td>
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<td>$C_{ls}$</td>
<td>chloride concentration at surface of concrete ($\text{kg/m}^3$)</td>
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<td>$D$</td>
<td>diameter of the steel reinforcing bar (mm)</td>
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<td>$D_a$</td>
<td>chloride diffusion ($\text{cm}^2/\text{year}$)</td>
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<td>effective depth to compression reinforcement (mm)</td>
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<td>$d$</td>
<td>depth from compression face to centroid of all tensile reinforcements &gt; $0.8h$ (mm)</td>
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<td>$E_c$</td>
<td>modulus of elasticity of concrete (MPa)</td>
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<td>$E_{cd}$</td>
<td>modulus of elasticity of the deck concrete (MPa)</td>
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<td>$E_{ci}$</td>
<td>modulus of elasticity of concrete at transfer (MPa)</td>
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<td>$E_p$</td>
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<td>$e_d$</td>
<td>eccentricity of the deck with respect to the gross composite section (mm)</td>
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\( e_e \) eccentricity of the centroid of the strands at the end of the girder with respect to the centroid of the gross section. Unbonding is neglected (mm)

\( e_g \) distance between centers of gravity of the non-composite beam (mm)

\( e_m \) eccentricity of the centroid of the strands at mid-span with respect to the centroid of the gross section (mm)

\( F \) Faraday’s constant

\( f_{cgp} \) the concrete stress at the c.g. of prestressing tendons due to prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (MPa)

\( f_{pt} \) stress in prestressing strands immediately after transfer, taken not less than 0.55 \( f_{py} \)

\( f_{pu} \) stress in prestressing steel at nominal (MPa)

\( f_s \) stress in mind steel tension reinforcement at nominal (MPa)

\( f'_c \) specified compressive strength of concrete at 28 days (MPa)

\( f'_c' \) compressive strength of concrete (MPa)

\( f'_s \) stress in mind steel compressive reinforcement at nominal (MPa)

\( f_{py} \) yield strength of prestressing (MPa)

\( h_f \) compression flange depth of member (mm)

\( I \) the current (A)

\( I_g \) gross moment of inertia, non-composite girder (mm\(^4\))

\( K_L \) factor for the type of prestressing steel used 30 for low relaxation strands and 7 for other prestressing steel

\( K_{df} \) transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in the section being considered for the time period between deck placement and final time

\( K_{id} \) transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in the section being considered for the time period between transfer and deck placement

\( K_1 \) correction factor for source of aggregate to be taken as 1.0 unless determined
k  coefficient of carbonation (mm/√years)
L  length of girder (mm)
L_{db}  average unbonded length of the unbonded strands (mm)
L_{t}  transfer length of prestressing strands (mm)
l  span length (m)
M  the atomic mass of the metal (56 g for Fe)
M_{loss}  the mass of steel lost in time (g) to form rust
m_{1}  the percentage of steel mass loss
N_{b}  number of beams
P  prestressing force (N)
Q  load effect
RH  relative humidity (%) by 45% ≤ RH ≤ 80%
R_{n}  component resistance
S  girder spacing (mm)
s  spacing of stirrups (mm)
T_{sp}  time of corrosion induced concrete spalling (years)
t_{d}  age of deck placement (days)
t_{cr}  time of corrosion induced concrete cracking (years)
t_{r}  lifetime of concrete (years)
t_{s}  slab thickness (mm)
t_{ser}  time to severe cracking
t_{1st}  time to initiation cracking
V_{c}  shear resistance in concrete (N)
V_{n}  shear resistance of section (N)
V_{p}  shear resistance in prestressing force (N)
V_{s}  shear resistance in shear reinforcement (N)
V/S  volume – surface ratio (mm) by 100mm ≤ V/S ≤ 300mm
W  unit water content (kg/m^3) by 130kg/m^3 ≤ W ≤ 230 kg/m^3
W/C  water to cement ratio
w_{g}  uniformly distributed girder self-weight
W'_{steel,cr}  the critical mass loss of steel g/cm^2
\(X_c\) carbonation depth at considering time (mm)

\(X_h\) distance from harp point to center of span

\(y_{pse}\) distance from centroid of prestressing steel to extreme bottom fibers of the section, calculated at the end of the member

\(y_{psm}\) distance from centroid of prestressing steel to extreme bottom fibers of the section calculated at mid span

\(z\) the ionic charge (2 For Fe)

\(\Delta f_{cd}\) change in concrete stress at the centroid of the strands due to long term losses between transfer and deck placement, including of deck weight and superimposed loads (MPa)

\(\Delta f_{cdf}\) change in concrete stress at the centroid of prestressing strands due to shrinkage of deck concrete (MPa)

\(\Delta f_{pCD}\) loss of prestress due to creep after deck placement (MPa)

\(\Delta f_{pCR}\) loss of prestress due to creep (MPa)

\(\Delta f_{pES}\) loss of prestress due to elastic shortening (MPa)

\(\Delta f_{pRI}\) loss of prestress due to relaxation of prestressing steel before transfer (MPa)

\(\Delta f_{pR2}\) loss of prestress due to relaxation of prestressing steel after transfer (MPa)

\(\Delta f_{pSD}\) loss of prestress due to concrete shrinkage after deck placement (MPa)

\(\Delta f_{pSR}\) loss of prestress due to concrete shrinkage (MPa)

\(\Delta f_{pSS}\) gain of prestress due to shrinkage of deck concrete (MPa)

\(\Delta LL\) deflection due to live load of girder (mm)

\(\Delta ps,t\) deflection due to prestressing girder (mm)

\(\Delta sw,t\) deflection due to girder self-weight (mm)

\(\Delta t\) total deflection of girder (mm)

\(\varepsilon_{bdf}\) concrete shrinkage strain of girder after to deck placement \((x \times 10^{-6})\)

\(\varepsilon_{bd}\) concrete shrinkage strain of deck after placement\((x \times 10^{-6})\)

\(\varepsilon_{b}\) concrete shrinkage strain of girder before to deck placement \((x \times 10^{-6})\)

\(\varepsilon_{as}(t_0)\) final autogenous shrinkage at \(t_0\) \((\times 10^6)\) in table 11 of DPT 1332 -2007

\(\propto 1\) 1.0 for dry surface concrete and 0.95 for wet surface concrete
$\alpha_2$  coefficient of environmental carbonation

$\gamma_c$  unit density of concrete (kg/m$^3$)

$\gamma_i$  1.0 for structures that require a lifetime of non-maintenance less than 15 years.

1.1 for structures that require a lifetime of non-maintenance more than 15 years.

$\gamma$  load factor

$\lambda$  coefficient for type of concrete

$\phi$  resistance factor

$\Psi_b$  girder creep coefficient
Chapter 1
Introduction

1.1 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 bridges which are a component part of construction in expressway and elevated train structure in Bangkok and surrounding areas. As the prestressed concrete bridges have become aging structure, maintenance of this structural is specific techniques. It have been reported that the prestressed concrete bridge girders is workable but in unique repair method (Kasan, 2009).

Their repair methods needs to be considered for efficient utilizing the budget with the condition such as structural performance, safety to user, socials or commercials, environment and limited budget on each annual 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 that well known on BRIDGIT and Pontis system in The United States of America. The BMSs has classified into three parts that comprise in assessing bridge condition, modeling deterioration prediction and the decision to the maintenance of bridges.

1.2 Statement of problem

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. In consequence of prestressed concrete bridges structure is popular in
Thailand. However, few studies have been developed in actual inspection results for evaluation and prediction of prestressed concrete bridges. Moreover, prestressed concrete behavior is different from reinforcement concrete such as high strength of materials and loss of prestress that need to be studied. Therefore, studying the deterioration prediction of prestressed concrete bridge girder is required.

1.3 Objectives and scope of study

The objective of this study is to develop the software for predicting deterioration of prestressed concrete bridge girder. The software represents a tool for maintenance planning which can be applied to reduce maintenance cost and extend service life of bridges. The scope of this study is presented in Figure 1.1.

The other objectives of this study are the following:
- Determine main parameters affecting performance of prestressed concrete and need to be assessed
- Determine the deterioration models and structural performance
- Determine reliability of the structure based on probability of failure
- Design database to accumulate the information such as inspection data, technical specification data and results on each criteria
- Support maintenance works of infrastructure
Figure 1.1 Scope of this study
Chapter 2  
Literature Review 

2.1 General 

To understand deterioration prediction of prestressed concrete bridge girder based on probability of failure. The important topics of this study was focused on deterioration of steel corrosion in prestressed concrete, bridge management, fundamental requirement performance of prestressed concrete bridge, deterioration prediction model and a probabilistic approach for modeling prediction. The literature review is represented in the following subjects.

2.2 Deterioration of steel corrosion in prestressed concrete 

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 surrounding environment. In order to understand the effectiveness of steel corrosion in concrete due to performance of concrete structures, a basic process and deterioration mechanism of steel corrosion in concrete is necessary.

It has been wildly reported that the passive oxide film on the surface of steel will be inhibited steel corrosion in concrete due to surrounding environment. However, there have several kinds of factor to destroy the passive oxide film depend on their parameter such as material properties both of steel and concrete, porosity or permeability in concrete, thickness of covering concrete, temperature, relative humidity, especially on penetration of carbon dioxide and chloride (Ferreira, 2004). The process of corrosion of steel in concrete is divided into an initiation state and propagation stage. In initiation state, it is well known that mostly occur since either carbon dioxide or chloride attack into concrete until the passive oxide layer of steel will be absented. Subsequently, The propagation stage, the passive oxide film have been destroyed. It is mean that, staring of steel corrosion in concrete from this stage
The process of corrosion of steel in concrete is shown in Figure 2.1.

![Figure 2.1 The process of corrosion of steel in concrete (Siemes et al., 1999)](image)

Considerably influence structural reliability during decreased the capacity of structure is steel corrosion in concrete. Because of corrosion of steel in concrete leading to steel section loss and increasing the volume of corrosion product that allow concrete to cracking. Then, losing of bond between concrete and steel can lease spalling or delamination of concrete that significantly affect to increase corrosion activity (West. et al. 1999).

According to steel corrosion in concrete structure on difference type of steel, ACI 222.2R-2001 reported that in case of steel in concrete both of reinforcing and prestressing steel are similarly corroded in term of percentage loss of section steel. However, In case of prestressed concrete structure, prestressing strand is more serious than reinforcing steel. Because of the failure due to prestressing strand can lead to collapse suddenly, in term of stress in prestressing strand exceed the limitation of allowable stress that occurs from corrosion (about 55% to 65% of its ultimate tensile
strength). There are two types of prestressed concrete: pretensioned and post-tensioned.

- A pretensioned system, the tendons is completely surrounded by concrete of the member that means the prestressed steel is in intimate contact (bonded) with the concrete. Therefore, pretensioned system is similar to a typical corrosion of reinforced concrete.

- Post-tensioned systems, the tendons contained within a metal, plastic or paper duct. The duct is typically filled with cement grout, grease, or petroleum wax to provide corrosion protection for the steel. The most important of this system is the grout. If the tendon dusts are not fully filled with grout, corrosion is more susceptible. In addition, anchorage is a critical part of unbounded system which loss of anchor would cause in an effective loss of the entire tendon.

Furthermore ACI 222.2R-2001 reported that it not only breakdown on yielding of prestressing steel, other type of deterioration mechanisms in prestressing steel may inclined by high strength of prestressing steel. The pattern of individual corrosion form and stresses for several type of steel may trigger hydrogen induced embrittlement, corrosion fatigue and stress corrosion cracking (SCC). It complex to identify and no indicate or slight of signal due to prestressing steel collapse by these individual corrosion.

2.2.1 Case studies of corrosion deterioration due to prestressed concrete bridges

Goin (2000) reported that the Lowe’s walkway bridge across to motor speedway in North Carolina which is prestressed concrete structure was collapsed causing corrosion in prestressing steel. According to inspections problem, can be detected as corrosion product around the prestessing strand in the bridge causing of bridge structure failure on that time. Calcium chloride which is cause of corrosion due to chloride attack was founded in grout around of prestressing steel. The disaster of bridge structure was caused by 11 prestressing steel in concrete were collapse by
corrosion. Also, because of that bridge is private structure which was lacked of inspection for check the possibility of any damage from the government office. In this case the sign of corrosion deterioration of bridge may have been exposed, if the bridge had been carefully investigated. The bridge structure collapse due to corrosion failure is shown in Figure 2.2.

![Figure 2.2 Bridge structure collapse due to corrosion failure (Goin, 2000)](image)

According to deterioration of bridge structure due to several cases, especially from steel corrosion in concrete were needed to inspecting of structure to prevent any damages. Bruce et al (2008) has been investigated the prestressed concrete bridge that is Humanatua Stream Bridge in New Zealand. The Humanatua Stream Bridge was constructed in 1966, the inspection consequence was reported that it has been faced about corrosion problem in the propagation stage. Cracked and spalled the cover concrete were caused by corrosion failure due to this propagation stage as shown in Figure 2.3 but prestressing strand still has not yet collapsed. Also, summarize that significances of corrosion in prestressing strand can be terrible while the stirrups does not afford an initial signal of impending corrosion of the prestressing strand.
Additional case studied, Taffe et al. (2010) and Torill et al. (2012) were studied on the performance of old prestressed concrete bridge structure. First reference was studied the condition assessment of 45-year old prestressed concrete bridge by using non-destructive method. Spandauer-Damm-Brücke in Berlin-Charlottenburg is investigated bridge structure which was crossing the railway. The bridge structure was established between 1960 and 1963. According to inspection, the results of condition assessment can summarize that it has been confronted about corrosion in bridge structure. Corrosion of steel in concrete both of reinforcing and prestressing steel as shown in Figure 2.4. Delamination and fracture in prestressing steel were founded in this bridge structure. Causing of this bridge is corroded of steel due to de-icing salts. In the report on last reference has also investigated the performance of 45-year-old corroded in prestressed concrete bridge structure. The bridge structure was constructed in 1957 which is the first prestressed construction bridge in Australia. In the same way of corrosion problem, the bridge structure located near to marine environment. Calcium chloride which is cause of corrosion due to chloride damage was founded in grout around of prestressing steel on particular beam. Some part of bridge structure due chloride corrosion in prestressing steel is shown in Figure. 2.5. Nevertheless, other beam corrosion deterioration until fracture of prestressing steel of this inspection, not only chloride attack into the concrete structure, it is may proposed from discussing on other reason of failure such as stress corrosion-cracking and hydrogen embrittlement which are complicated corrosion mechanism also.
In the conclusion, corrosion damage of prestressed concrete can be classified into three types.

- Minor corrosion, loss of cross-sectional area result in induced stress corrosion cracking and relaxation of prestressing strands.
- Severe corrosion, the increasing of rust product induced to concrete cracking. Then, losing of bond between concrete and steel can induced spalling of concrete cover or delamination that extremely affect to increase corrosion activity.
- Fracture, damage from over allowable stress, hydrogen embrittlement and stress-corrosion cracking can significantly affect capacity of structure.

Subsequently, corrosion deterioration in prestressed concrete can summarized that according to similar influences are occupied in to steel corrosion in concrete on both of prestressing and reinforcing steel. The failures of prestressing steel are difference from reinforcing steel, some kind of failure due to prestressing steel may no sign of warning to collapse even through corrosion on reinforcing steel such as stirrups were founded. In additional, corrosion is not as well documented in prestressed concrete structures as in reinforced concrete structures.

2.3 Bridge management system

Miyamoto et al (2001) presented the background of bridge management systems that in Japan, they need to designed the bridge management systems according to repair of bridge structure has become their main interested decisions. Because of the number of deterioration of bridge structures has slightly increased. Similarly to Standard specification for concrete structure Maintenance, JSCE (2007) reported that infrastructure in Japan has constructed in the previous middle of the 20th century. Increasing a number of damage bridges, there need systemically to rehabilitation the structure due to technological knowledge in concrete. In order to ensure safety and service life of bridge structures, necessity of maintenance planning is required.

To operate a system of bridge structures under the limitation of budget and maintenance, Bridge Management Systems (BMSs) are designed. Bridge Management Systems have been established to administer bridge structures organization due to the assignment of operating several of bridge structures has change to progressively deterioration on structures (Elbehair, 2007).

The components of bridge management systems are described in the same way process on several references. The fundamental of bridge management systems components can simply divide into: database of bridge, deterioration model, finance
model and optimization model are proposed by guidelines for bridge management systems AASHTO (2001). Likewise to Miyamoto et al (2001) reported that the elementary of BMSs which is Japanese bridge management systems (J-BMSs) are divided into bridge inventory, evaluation of performance, deterioration prediction, effect of maintenance including finance of maintenance and optimization of rehabilitation planning as shown in Figure 2.6.

- **Evaluation of performance**, identify the existing performance of the bridge, compared to its performance at the time of construction. Commonly, the performance of the bridge is measured by an investigation which including document search, inspection and testing. The regular investigations of bridges are necessary for alerting bridge engineers to the deterioration of the bridge for several of reasons. Furthermore, inspections also support bridge engineers to determine future maintenance requirements.

- **Bridge deterioration prediction**, as results of assessment of structural performance, the deterioration can be identified and the remaining life of structure can be determined using the predicted models of deterioration. Through, deterioration progress shall be appropriately predicted to understand the deterioration mechanism and using a correctly model. However, bridge deterioration can be estimated the condition from normal condition excluding an extremely events such as seismic or fire.

- **Cost and effect of maintenance**, the cost of maintenance, repair and rehabilitation can be estimated as unit cost or percentage cost of initial constructed. In case of effect of maintenance, most BMSs assume the performance after repair is the same as the bridge was constructed.
- **Optimization of rehabilitation strategy**, the priorities for operation the activity for the maintenance is considered. The cost of maintenance spends most of the available funding. Therefore, the budget for these activities should be carefully allocated, especially when the life cycle cost is decided. Setting priorities for maintenance activities is a multiple-criteria decision-making problem which needs evaluation function at the same time (i.e., which bridge to repair and which repair techniques).
According to study the practices of Bridge System Management by Minchin et al. (2006) regarding that there many BMSs program were used on each state highway agencies in the United Sates. In order to optimization the maintenance planning of bridge structures for preserve the performance of structure and make sure that safe condition. Pontis and BRIDGIT are BMSs programs which well known in the United State. Pontis was established in the primary 1990s for Federal highway Administration (FHwA) and developed to American Association of state Highway and Transportation Officials (AASHTO) product in 1994 while BRIDGIT was established in 1985 through the Nation Engineering Technology Cooperation and National Cooperative Highway Research Program (NCHRP). The component of both program are similarly process which are bridge inventory, inspection data, simulating bridge structure performance at the current and future, finance modeling and solving the balance of situation in optimizing models. However, a few states has to developed individual BMSs program for convenient functions and qualities such as Pennsylvania BMSs, North Carolina BMSs, Indiana BMSs and other BMSs research which have to develop with universities and local agencies.

In summary, mostly BMSs has been developed for evaluating the performance of structure based on visual and condition rating in reinforced concrete structure. In case of prestressed concrete, only visual inspection and condition rating maybe not enough for evaluation because of prestressed concrete has more serious consequences than in reinforced concrete. Consequently, non-destructive and partially-destructive tests shall be applied for more accuracy evaluation in this research.

2.4 Fundamental requirement performance of prestressed concrete bridge girder

It has widely known that concrete and steel were the composited of reinforced concrete. General of concrete mechanism is strong in compression but weak in tension, Steel is forceful in both of compression and tension. For this reason, reinforced concrete used concrete to oppose compression and used steel to oppose in tension. While, flexural cracks are arise during applied the load due to low tensile capability. In order to avoid or reduce cracks from this processing, compression force is applied to the reinforced concrete segment. Initially imposed force can called a
prestressing forces, the influence of prestressing is to decrease the tensile stress to be lower than cracking stress. Thus, meaning of flexural cracks during reinforcing will be lost. Normally the system of prestressed concrete divided to pre tension and post tension. For pre tension is achieved concrete in a factory by casting the concrete in molds with tendon applied the tension force. Then prestressing steel are cut after concrete has been enough cured. On the other hand, for post tension is performed concrete in the filed by casting in place and applied the tension force after concrete has been enough cured (Naaman, 2005).

2.4.1 Design code for prestressed concrete

In the United States, the specification standard for prestressed concrete is supported by agencies such as Precast/Prestressed Concrete Institute (PCI) and Post-Tensioning Institute (PTI). American Concrete Institute (ACI) is also provided the standard of prestressed concrete on Building Code Requirements for Structural Concrete (ACI 318) in chapter 18.

In particular, American Association of state Highway and Transportation Officials (AASHTO) is presented The AASHTO LRFD Bridge Design Specification to bridge design. Several chapters of AASHTO LRFD Bridge Design Specification are related to design of bridge. In addition Thailand’s Department of Highway (DOH) is also refers to AASHTO LRFD Bridge Design Specification.

2.4.2 Material of properties

According to AASHTO (2007) regarding to the materials of properties for prestressed concrete both of concrete and prestressing steel. In order to designs performance of prestressed concrete, materials of properties were required that conform to construction specifications standard.

2.4.2.1 Concrete

The mechanical properties of concrete, the fundamental to prestressed concrete design were including of compressive strength, modulus of elasticity and
modulus of rupture. Basically of prestressed concrete typically need higher strength than reinforced concrete that relevant to increasing of allowable stress and reducing of creep and shrinkage in concrete. In particular, the ranges of compressive strength of prestressed concrete is between 28 – 70 MPa at 28 days which difference from reinforced concrete that is between of 16 – 70 MPa at 28 days. Subsequently, modulus of elasticity \((E_c)\) that related to density concrete between 1440 and 2500 (kg/m\(^3\)) defined as shown in equation (2.1). The following equation (2.2) describes for normal unit density of concrete (2320 kg/m\(^3\)) (AASHTO, 2007).

\[
E_c = 0.043 K_1 \gamma_c^{1.5} \sqrt{f'_c}
\]  

(2.1)

\[
E_c = 4800 \sqrt{f'_c}
\]  

(2.2)

where, \(K_1\) is correction factor for source of aggregate to be taken as 1.0 unless determined, \(\gamma_c\) is unit density of concrete (kg/m\(^3\)) and \(f'_c\) is compressive strength of concrete (MPa)

### 2.4.2.2 Prestressing steel

There several types of prestressing tendons such as strand, wires, round bar or threaded rods. The traditional type of prestressing tendons is a wire strand likewise to Thailand’s prestressed concrete construction also. Mechanical properties of prestressing tendons were described on tensile strength, yield strength and modulus of elasticity for design prestressed concrete structures. The properties of prestressing strand are reported in table 2.1. The modulus of elasticity for prestressing tendons \((E_p)\) are 197 000 MPa and 207 000 MPa for strand and bar respectively (AASHTO, 2007). Because of prestressing strand produced from winding of small wires collectively that have an influence to lower of modulus of elasticity than steel bar (Naaman, 2005). According to a twisting of small wires together, it’s mean that section area of prestressing strands will be less than steel bar a little bit. So a typical characteristic of stress-relieved prestressing strand is shown in table 2.2.
Table 2.1 Properties of prestressing strand and bar

<table>
<thead>
<tr>
<th>Material</th>
<th>Grade or type</th>
<th>Diameter (mm)</th>
<th>Tensile strength, $f_{pu}$ (MPa)</th>
<th>Yield strength, $f_{py}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strand</td>
<td>1725 MPa (Grade 250)</td>
<td>6.35 to 15.24</td>
<td>1725</td>
<td>85% of $f_{pu}$, except 90% of $f_{pu}$ for low-relaxation strand</td>
</tr>
<tr>
<td></td>
<td>1860 MPa (Grade 270)</td>
<td>9.53 to 15.24</td>
<td>1860</td>
<td></td>
</tr>
<tr>
<td>Bar</td>
<td>Type 1, plain</td>
<td>19 to 35</td>
<td>1035</td>
<td>85% of $f_{pu}$</td>
</tr>
<tr>
<td></td>
<td>Type 2, Deformed</td>
<td>19 to 35</td>
<td>1035</td>
<td>80% of $f_{pu}$</td>
</tr>
</tbody>
</table>

Source: AASHTO (2007)

Table 2.2 A typical characteristic of stress-relieved prestressing strand

<table>
<thead>
<tr>
<th>Prestressing steel</th>
<th>ASTM type or Grade</th>
<th>Nominal diameter (mm)</th>
<th>Nominal area ($\text{mm}^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stress-relieved 7-wire strands (ASTM A416)</td>
<td>Grade 250</td>
<td>9.3</td>
<td>51.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10.8</td>
<td>69.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>12.4</td>
<td>92.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.2</td>
<td>139</td>
</tr>
<tr>
<td>Grade 270</td>
<td>9.5</td>
<td>54.8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>11.1</td>
<td>74.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>12.7</td>
<td>98.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>15.2</td>
<td>139</td>
<td></td>
</tr>
</tbody>
</table>

Source: TIS (1997)

2.4.3 Bridge design loadings

Bridges designed was needed to considering several loads on bridges in order to resist design loadings in a safe condition. Dead loads carry on structure all the time and perform to structure since construction. Live load was performed to bridges structure after finished construction due to service. Generally, loads on bridges can divided into a typical load and other loads. For typical load consist of dead load and live load. Other loads are fatigue, wind breaking load, centrifugal load, earth pressure, buoyancy, shrinkage, creep, temperature, stream flow, vehicle collision and earthquake (AASHTO, 2007).

Dead load that including of component of structure such as girder, slab, etc. Also non structure such as railings, signs, etc. It is can estimated the weight of dead load from the material’s density for instance concrete in normal weight, light weight concrete, steel and stone masonry are had density of materials in 2400, 1775 – 1925,
7850 and 2275 kg/m$^3$ respectively. Dead load of wearing surface is the directly mass of wearing surface that consist of wearing surface and utilities. Normally of wearing surface was used asphalt which approximately had density of 2250 kg/m$^3$ and thickness of asphalt around 5 – 10 cm (AASHTO, 2007).

![Figure 2.7 Characteristics of the design truck (HL-93)](image)

Live load is a temporary load perform to structure, that are need to concerned such as dynamic load or impact load due to movement of vehicle. According to AASHTO (2007) described the type of standard live load called the HL-93 loading (standard for highway loading, 1993) to design bridge structure into truck load (HL-93), tandem load and uniform lane load as shown in Figure 2.7. The concept of simple span design of live load is the maximum effect of between combinations of truck load (HL-93) including uniform lane load and tandem load including uniform lane load. However, the maximum effect of flexural and shear are depended on the position of live load also.

Impact due to live load is dynamic load, occurring from movement of vehicle. Considering impact force shall be concerned the percentage of increasing of live load from equation (2.3), where L is length of girder.
Impact factor, \( I = \frac{15.24}{L+38} \leq 0.3 \) \hspace{1cm} (2.3)

2.4.3.1 Distribution of Live Load to Girder

According to AASHTO (2007) give the reasons for distribution of live load to girder, that is normal for bridge to have more than one girder in the structure. So it is needed to distributed load to each girder. There are two methods; 1. Using AASHTO formulation for designing by approximates value 2. Using of finite element method for refined analysis. Fundamental of design of distribution of live load to girder are distributed load from lane load to girder load both of flexural and shear called distribution factor (DFs). The interior I- girder of prestressed concrete as shown in table 2.3 both of flexural and shear.

<table>
<thead>
<tr>
<th>Distribution factor</th>
<th>One design lane load</th>
<th>Two or more design lane load</th>
<th>Range of applicability</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Moment</strong></td>
<td>0.06 + ( \left( \frac{S}{4300} \right)^{0.4} ) ( \left( \frac{S}{L} \right)^{0.3} ) ( \frac{K_g}{L_{c3}} ) ( 0.1 )</td>
<td>0.075 + ( \left( \frac{S}{2900} \right)^{0.6} ) ( \left( \frac{S}{L} \right)^{0.2} ) ( \frac{K_g}{L_{c3}} ) ( 0.1 )</td>
<td>1100 ( \leq S \leq 4900 ) ( 110 \leq t_s \leq 300 ) ( 6000 \leq L \leq 73000 ) ( N_b \geq 4 ) 1100 ( \leq K_g \leq 4900 )</td>
</tr>
<tr>
<td><strong>Shear</strong></td>
<td>0.36 + ( \left( \frac{S}{7600} \right) )</td>
<td>0.2 + ( \left( \frac{S}{3600} \right) - \left( \frac{S}{10700} \right)^2 )</td>
<td>1100 ( \leq S \leq 4900 ) ( 6000 \leq L \leq 73000 ) ( 110 \leq t_s \leq 300 ) ( N_b \geq 4 )</td>
</tr>
</tbody>
</table>

Source: AASHTO (2007)

where, \( S \) is girder spacing (mm), \( L \) is span length (mm), \( t_s \) is slab thickness (mm) and \( N_b \) is number of beams. While, \( K_g \) can calculated from \( n \) \( I_g + A_g e_g \) by \( n \) is modulus of elasticity of beam \( (E_c, \text{beam}) \) divided modulus of elasticity of slab \( (E_c, \text{slab}) \). And \( I_g \) is gross moment of inertia, non-composite girder \( (\text{mm}^4) \), \( A_g \) is gross area, non-composite girder \( (\text{mm}^2) \) and \( e_g \) is distance between centers of gravity of the non-composite beam (mm).
2.4.4 Load and Resistance factor

General, philosophies of design can divided into allowable stress design (ASD), load factor design (LFD) and load and resistance factor design (LRFD). AASHTO (2007), is directly indicated what type of design which is conforming to Thailand Department of Highway (DOH) designs bridge construction. The idea this standard design is factored load should less than or equal to factored resistance. A design criterion in AASHTO LRFD is shown in equation (2.4).

\[ \sum \gamma Q \leq \bar{R}_n \quad (2.4) \]

where, \( Q \) is load effect, \( R_n \) is component resistance, \( \gamma \) is load factor and \( \bar{R} \) is resistance factor. For load factors and load combinations have 4 types in the following and specified in table 2.4.

- **Ultimate limit state** is related to strength of structure that containing of strength I (basic load without wind), II (special design vehicles load without wind), III (wind in excess of 90 km/hr.), IV (DL/LL > 7) and V (wind of 90 km/hr.).

- **Extreme event limit state** involving structure when confront to natural disaster or unexpected calamity such as earthquake, flood, or collision that consist of extreme event I (earthquake) and II (collision).

- **Serviceability limit state** is related to operating conditions of structure involving to stress, deflection and crack width that consist of service I (normal operation with 90 km/hr.), II (intended to control yielding of steel structure) and III (longitudinal analysis relating to tension in prestressed concrete superstructure).

- **Fatigue limit state** is the limitation of stress due to repetitive load on the bridge construction.
Table 2.4 Load Factors and Load Combinations

| Load Combination Limit state | DC | DD | DW | EH | EV | ES | EL | LL | IM | CE | BR | PL | LS | WA | WS | WL | FR | TU | CR | SH | TG | SE | EQ | IC | CT | CV |
|------------------------------|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| STRENGTH I (unless noted)    | $\gamma_p$ | 1.75 | 1.00 | - | - | 1.00 | - | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| STRENGTH II                  | $\gamma_p$ | 1.35 | 1.00 | - | - | 1.00 | - | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| STRENGTH III                 | $\gamma_p$ | - | 1.00 | 1.40 | - | 1.00 | - | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| STRENGTH IV                  | $\gamma_p$ | - | 1.00 | - | - | 1.00 | - | 0.50/1.20 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| STRENGTH V                   | $\gamma_p$ | 1.35 | 1.00 | 0.40 | 1.0 | 1.00 | - | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| EXTREME EVENT I              | $\gamma_p$ | $\gamma_{EQ}$ | 1.00 | - | - | 1.00 | - | - | - | 1.00 | - | - | - | 1.00 | - | - | - | - | - | - | - | - | - | - | - | - | - |
| EXTREME EVENT II             | $\gamma_p$ | 0.50 | 1.00 | - | - | 1.00 | - | - | - | - | - | - | 1.00 | 1.00 | 1.00 | - | - | - | - | - | - | - | - | - | - | - | - | - |
| SERVICE I                    | 1.00 | 1.00 | 1.00 | 0.30 | 1.0 | 1.00 | 1.00/1.20 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| SERVICE II                   | 1.00 | 1.30 | 1.00 | - | - | 1.00 | 1.00/1.20 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| SERVICE IIII                 | 1.00 | 0.80 | 1.00 | - | - | 1.00 | 1.00/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| SERVICE IV                   | 1.00 | - | 1.00 | 0.70 | - | 1.00 | 1.00/1.20 | - | 1.0 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| FATIGUE - LL,IM & CE ONLY    | - | 0.75 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |

Source: AASHTO (2007)

The following permanent and transient loads and forces shall be considered:
- DD = Downdrag
- DC = Structural Components and Attachments
- DW = Wearing Surfaces and Utilities
- EH = Horizontal Earth Pressure
- EV = Vertical Pressure of Earth Fill
- ES = Earth Surcharge Load
- BR = Veh. Braking Force
- CE = Veh. Centrifugal Force
- CR = Creep
- CT = Veh. Collision Force
- CV = Vessel Collision Force
- IC = Ice Load
- PL = Pedestrian Live Load
- SE = Settlement
- SH = Shrinkage
- TG = Temperature Gradient
- TU = Uniform Temperature
- WA = Water Load
- WL = Wind on Live Load
- WS = Wind Load on Structure
- EQ = Earthquake

Ref. code: 25595622040680JPL
Table 2.5 Resistance factors for concrete

<table>
<thead>
<tr>
<th>Types</th>
<th>Resistance factor (Ø)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexural and Tension</td>
<td></td>
</tr>
<tr>
<td>in Reinforced concrete</td>
<td>0.90</td>
</tr>
<tr>
<td>in Prestressed concrete</td>
<td>1.00</td>
</tr>
<tr>
<td>Shear in normal weight concrete</td>
<td>0.90</td>
</tr>
<tr>
<td>Axial compression</td>
<td>0.75</td>
</tr>
<tr>
<td>Bearing on concrete</td>
<td>0.70</td>
</tr>
</tbody>
</table>

Source: AASHTO (2007)

For resistance factors Ø are unique for different kinds of action such as flexural moment or shear, etc. and different kinds of materials. Especially, concrete structure are described specified on each type (AASHTO LRFD 5.5.4.2) as shown in table 2.5

2.4.5 Flexural Strength of Prestressed concrete Girder

Flexural resistance at the strength limit state shall be calculated with the equation accordance to AASHTO (2007). For the ultimate flexural strength $M_n$ can express for flanged sections as shown in equation (2.5).

$$M_n = A_{ps} f_{pu} (d_p - a/2) + A_s f_s (d_s - a/2) - A_s' f'_s (d'_s - a/2) + 0.85 f'_c (b - b_w) h_f (a/2 - h_f/2)$$  (2.5)

where:

- $A_{ps}$ = area of prestressing steel (mm$^2$)
- $f_{pu}$ = stress in prestressing steel at nominal (MPa)
- $d_p$ = effective depth to prestressing tendons (mm)
- $A_s$ = area of nonprestressed tension reinforcement (mm$^2$)
- $f_s$ = stress in mind steel tension reinforcement at nominal (MPa)
- $d_s$ = effective depth to mind steel tension reinforcement (mm)
2.4.6 Shear Strength of Prestressed concrete Girder

In sectional design methods, the shear capacity of prestressed concrete beam which is quite different from reinforced concrete that can acquire from concrete, steel stirrups and vertical component of prestressing force. As reported by AASHTO LRFD 5.8.3.3, the nominal shear resistance \( V_n \) will be expressed in equation (2.6).

\[
V_n = V_c + V_s + V_p
\]

(2.6)

in which:

\[
V_c = (0.05 \sqrt{f_c'} + 4.8 \frac{V_u d_p}{M_u}) b_w d
\]

by \( \frac{V_u d_p}{M_u} \leq 1 \) and \( 0.17 \lambda \sqrt{f_c'} b_w d_p \leq V_c \leq 0.42 \lambda \sqrt{f_c'} b_w d_p \) \hspace{1cm} (2.7)

\[
V_s = \frac{A_v f_y d_v \cot\theta}{s}
\]

(2.8)

\[
V_p = P \sin\theta
\]

(2.9)

where:

d = depth from compression face to centroid of all tensile reinforcements

> 0.8h (mm)

\( \lambda \) = 1.00 for normal weight concrete

0.85 for sand light-weight concrete

0.75 for all light-weight concrete

\( A_v \) = area of shear reinforcement within a distances (mm\(^2\))

\( d_v \) = effective shear depth (mm)
2.4.7 Allowable stress design

According to AASHTO (2007) regarding to the allowable stress at the service limits state of materials for prestressed concrete both of concrete and prestressing steel. In order to ensure the important limits state of prestressed concrete are concerned, allowable stress of concrete and prestressing force of tendons were required that conform to construction specifications standard.

2.4.7.1 Stress Limitations for Prestressing tendons

As reported by AASHTO LRFD 5.9.3, at the service limit state or the stress of prestressing tendons especially in pre-tensioning shall not exceed specifications as presented in table 2.6.

2.4.7.2 Stress Limitations for Concrete

Allowable stress in concrete are concerning of compressive stress and tensile stress in concrete. In case of allowable compressive stress in concrete is used to control creep that causes loss of prestress force by the time and allowable tensile stress in concrete is used to prevent cracking that reduces the practical section. For stress limitation, stress in concrete immediately after prestress transfer and stress in concrete at service loads are considered. On this structure condition is the existing structures so it means that allowable stress in concrete at service has a significantly considered. The stress in concrete at service state AASHTO (2007) allowance for stress shall not exceed the following:

(a) extreme fiber stress in compression due to prestress including sustained loads for 0.45 $f'_{c}$

(b) extreme fiber stress in compression due to prestress including total loads for 0.60 $f'_{c}$

\[ s = \text{spacing of stirrups (mm)} \]
\[ P = \text{prestressing force (N)} \]
(c) extreme fiber stress in tension due to prestressed tensile zone for 0.5
$$\sqrt{\frac{f_c'}{f_c}}$$

Table 2.6 Stress Limits for Prestressing Tendons

<table>
<thead>
<tr>
<th>Condition</th>
<th>Tendon Type</th>
<th>Stress-Relieved Strand and Plain High – Strength Bars</th>
<th>Low Relaxation Strand</th>
<th>Deformed High – Strength Bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-tensioning</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Immediately prior to transfer</td>
<td>0.70 ( f_{pu} )</td>
<td>0.75 ( f_{pu} )</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>At service limit state after all losses</td>
<td>0.80 ( f_{pu} )</td>
<td>0.80 ( f_{pu} )</td>
<td>0.80 ( f_{py} )</td>
<td></td>
</tr>
</tbody>
</table>

Source: AASHTO LRFD (2007)

2.4.8 Loss of prestress

A precise estimation of prestress loss is demanded efficient design of prestress concrete bridges. The loss of tensile stress in the prestress tendon that represents on the concrete constituent of the prestressed concrete section defined as the prestress losses (Jayaseelan and Russell 2007). AASHTO (2007) is defined the total loss of prestress \( \Delta f_{pLT} \) in articles 5.9.5.1 for prediction. They can be classified the prediction of losses to instantaneous losses and long term losses. Instantaneous losses are represented as anchorage set loss, frictional losses elastic shortening. Long term losses are considered of shrinkage, creep in concrete and relaxation of prestressing tendons as defined in article 5.9.5.2. They are two estimate methods of long term losses as approximate estimate of time dependent losses and refined estimates of time dependent losses. The approximate estimate of time dependent losses which is Lump Sum methods were suitable in the preliminary design as defined in article 5.9.5.3. The refined estimates of time dependent losses is calculated prestressing tendons stress
separately due to time dependent losses as defined in article 5.9.5.4 and shall be determined total prestress loss in equation (2.10).

\[
\Delta f_{pTL} = \Delta f_{pES} + (\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1})_{id} + (\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS})_{df}
\]  

(2.10)

where:

\((\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1})_{id}\) = sum of time dependent prestress losses between transfer and deck replacement (MPa)

\((\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS})_{df}\) = sum of time dependent prestress losses after deck replacement (MPa)

\(\Delta f_{pES}\) = loss of prestress due to elastic shortening (MPa)

\(\Delta f_{pSR}\) = loss of prestress due to concrete shrinkage (MPa)

\(\Delta f_{pCR}\) = loss of prestress due to creep (MPa)

\(\Delta f_{pR1}\) = loss of prestress due to relaxation of prestressing steel before transfer (MPa)

\(\Delta f_{pSD}\) = loss of prestress due to concrete shrinkage after deck placement (MPa)

\(\Delta f_{pCD}\) = loss of prestress due to creep after deck placement (MPa)

\(\Delta f_{pR2}\) = loss of prestress due to relaxation of prestressing steel after transfer (MPa)

\(\Delta f_{pSS}\) = gain of prestress due to shrinkage of deck concrete (MPa)

2.4.8.1 Elastic Shortening

The total elastic shortening losses in pretension member as defined in article 5.9.5.2.3 of the AASHTO (2007) can be computed in equation (2.11).

\[
\Delta f_{pES} = \frac{E_p}{E_{cl}} f_{cgp}
\]  

(2.11)
where:

- \( f_{cgp} \) = the concrete stress at the c.g. of prestressing tendons due to prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (MPa)
- \( E_p \) = modulus of elasticity of prestressing steel (Mpa)
- \( E_{ci} \) = modulus of elasticity of concrete at transfer (Mpa)

2.4.8.2 Shrinkage loss

The total shrinkage losses of girder concrete of the AASHTO (2007) refined methods is determined in two period that comprises of the loss occurring before to deck placement \( \Delta f_{PSR} \) and the loss occurring after deck placement \( \Delta f_{PSD} \) shall be determined in equation (2.12) and equation (2.14) respectively.

**Before to Deck Placement**

\[
\Delta f_{PSR} = \varepsilon_{bid} E_p K_{id}
\]  
(2.12)

in which:

\[
K_{id} = \frac{1}{1 + \frac{E_p}{E_{ci}} \frac{A_{ps}}{A_g} \left( 1 + \frac{A_g \frac{E_{pg}^2}{I_g}}{1 + \Psi_b (t_f, t_i)} \right) \left[ 1 + \Psi_b (t_f, t_i) \right]} 
\]  
(2.13)

where:

- \( \varepsilon_{bid} \) = concrete shrinkage strain of girder before to deck placement
- \( K_{id} \) = transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in the section being considered for the time period between transfer and deck placement
- \( \Psi_b (t_f, t_i) \) = girder creep coefficient due to loading at transfer conforms to article 5.4.2.3.2 of the AASHTO (2007)

**After Deck Placement**

\[
f_{PSD} = \varepsilon_{bdf} E_p K_{df}
\]  
(2.14)

in which:

\[
K_{df} = \frac{1}{1 + \frac{E_p}{E_{ci}} \frac{A_{ps}}{A_g} \left( 1 + \frac{A_g \frac{E_{pg}^2}{I_g}}{1 + \Psi_b (t_f, t_i)} \right) \left[ 1 + \Psi_b (t_f, t_i) \right]} 
\]  
(2.15)
where:

- \( \varepsilon_{\text{bdf}} \) = concrete shrinkage strain of girder after deck placement
- \( K_{\text{df}} \) = transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in the section being considered for the time period between deck placement and final time.

The concrete shrinkage strain is calculated by considering of humidity surrounding concrete structure, volume to surface area ratio and water to binder ratio etc. According to DPT 1332-2007 presented ultimate drying shrinkage strain and autogenous shrinkage strain that shall be determined in equation (2.16) and equation (2.19) respectively.

**Drying shrinkage**

\[
\varepsilon'_{\text{cs}}(t, t_0) = [1 - \exp\{-0.108(t - t_0)^{0.56}\}] \cdot \varepsilon'_{\text{sh}} \tag{2.16}
\]

in which:

\[
\varepsilon'_{\text{sh}} = -500 + 780(1 - \exp\left(\frac{\text{RH}}{100}\right)) + 380 \log e W - 50 \left[\log e \left(\frac{\text{V/S}}{10}\right)\right]^2
\tag{2.17}
\]

\[
t_0c \text{ and } t_c = \sum_{i=0}^{n} \Delta t_i \cdot \exp\left[13.65 - \frac{4000}{273 + T_i} \right] \tag{2.18}
\]

where:

- \( W \) = Unit water content (kg/m\(^3\)) by 130 kg/m\(^3\) ≤ \( W \) ≤ 230 kg/m\(^3\)
- \( \text{RH} \) = Relative humidity (%) by 45% ≤ \( \text{RH} \) ≤ 80%
- \( \text{V/S} \) = Volume – surface ratio (mm) by 100mm ≤ \( \text{V/S} \) ≤ 300mm
- \( T_i \) and \( T_0 \) = Temperature, by 0 - 40 Celsius

**Autogenous shrinkage**

\[
\varepsilon'_{\text{as}}(t, t_0) = \varepsilon'_{\text{as}}(t) - \varepsilon'_{\text{as}}(t_0) \tag{2.19}
\]

in which:

\[
\varepsilon'_{\text{as}}(t) = \gamma_b \varepsilon'_{\text{as\infty}} \left[1 - \exp\left(-a(t - t_s)^b\right)\right] \tag{2.20}
\]

\[
\varepsilon'_{\text{as\infty}} = 3070 \exp\{-7.2(W/C)\} \tag{2.21}
\]

where:

- \( \gamma_b \) = 1 for cement I
- \( W/C \) = water to cement ratio
\[ \varepsilon_{as}(t_0) = \text{final autogenous shrinkage at } t_0 (\times 10^{-6}) \text{ in table 11 of DPT 1332-2007} \]

a and b = multiples factors in table 12 of DPT 1332-2007

### 2.4.8.3 Creep loss

The total creep losses of girder concrete of the AASHTO (2007) refined methods is also determined in two periods that consist of the loss occurring before to deck placement \( \Delta f_{pCR} \) and the loss occurring after deck placement \( \Delta f_{pCD} \) shall be determined in equation (2.22) and equation (2.23) respectively.

**Before to Deck Placement**

\[
\Delta f_{pCR} = \frac{E_p}{E_{ci}} f_{cgp} \Psi_b(t_f, t_i) K_{id} \tag{2.22}
\]

where:

\( \Psi_b(t, t_i) \) = girder creep coefficient due to loading at transfer conforms to article 5.4.2.3.2 of the AASHTO (2007)

\( t_d \) = age of deck placement (days)

**After Deck Placement**

\[
\Delta f_{pCD} = \frac{E_p}{E_{ci}} f_{cgp} [\Psi_b(t_f, t_i) - \Psi_b(t_d, t_i)] K_{df} + \frac{E_p}{E_c} \Delta f_{cd} \Psi_b(t_f, t_d) K_{df} \tag{2.23}
\]

where:

\( \Delta f_{cd} \) = change in concrete stress at the centroid of the strands due to long term losses between transfer and deck placement, including of deck weight and superimposed loads.

### 2.4.8.4 Relaxation of presstressing strands

Similar to the other long term losses, the relaxation of prestressing strands loss of the AASHTO (2007) refined methods is also calculated in two periods, however there are equivalent determination of both relaxation before and after deck placement which represented as equation (2.24).
\[
\frac{f_{pt}}{K_L} \left( f_{pt} - 0.55 \right) = 0.29
\]

where:
- \( f_{pt} \) = stress in prestressing strands immediately after transfer, taken not less than 0.55 \( f_{py} \)
- \( K_L \) = 30 for low relaxation strands and 7 for other prestressing steel

### 2.4.8.5 Shrinkage of deck concrete

In the composite section, deck concrete has shrinks which resulting to increase in the prestressing force. On the article 5.9.5.4.3d of the AASHTO (2007) has represented as equation (2.25).

\[
\Delta f_{pSS} = \frac{E_p}{E_c} \Delta f_{cdf} K_{df} \left[ 1 + 0.7 \Psi_b \left( t_f, t_d \right) \right]
\]

(2.25)

in which:
- \( \Delta f_{cdf} \) = change in concrete stress at the centroid of prestressing strands due to shrinkage of deck concrete (MPa)
- \( \varepsilon_{bdf} \) = concrete shrinkage strain of deck after placement
- \( A_d \) = area of deck concrete (mm\(^2\))
- \( E_{cd} \) = modulus of elasticity of the deck concrete (MPa)
- \( e_d \) = eccentricity of the deck with respect to the gross composite section (mm)
- \( \Psi_b \left( t_f, t_d \right) \) = girder creep coefficient due to loading at final time conforms to article 5.4.2.3.2 of the AASHTO (2007)

### 2.4.9 Deflection

Deflection and camber of prestressed concrete girder determinations intend to check on dead load, live load, prestressing force and the time dependent loss of concrete creep, shrinkage and relaxation of strands. As AASHTO (2007)
specifications do not identify a formula to predict the deflection, especially the deflection due to prestress losses, there have been some researches (Rizkalla et al, 2011 and Rosa et al, 2007) studies to propose the prediction of deflection due to prestressing. The total deflection can be express in equation (2.27).

\[ \Delta_t = \Delta_{ps,t} - \Delta_{sw,t} - \Delta_{LL} \] (2.27)

where:

- \( \Delta_t \) = total deflection of girder (mm)
- \( \Delta_{ps,t} \) = deflection due to prestressing girder (mm)
- \( \Delta_{sw,t} \) = deflection due to girder self-weight (mm)
- \( \Delta_{LL} \) = deflection due to live load of girder (mm)

The deflection due to self-weight of girder \( \Delta_{sw,t} \) is given in equation (2.28)

\[ \Delta_{sw,t} = \frac{5w_g L^4}{384E_{ci} I_g} \] (2.28)

where:

- \( w_g \) = uniformly distributed girder self-weight
- \( L \) = girder length
- \( I_g \) = moment of inertia of the girder cross section

The deflection due to prestressing girder \( \Delta_{ps,t} \) is taken as equation (2.29).

\[ \Delta_{ps,t} = \Delta_{ps,i} - \frac{P_i - P_t}{E_{ci} + E_c I_g} \left( \frac{e_m L^2}{8} - \left( e_m - e_e \right) \frac{(L/2 - X_h)^2}{6} - \frac{e_m (L_{db} + L_t)^2}{6} \right) \] (2.29)

in which:

\[ \Delta_{ps,i} = \frac{P_i}{E_{ci} I_g} \left( \frac{e_m L^2}{8} - \left( e_m - e_e \right) \frac{(L/2 - X_h)^2}{6} - \frac{e_m (L_{db} + L_t)^2}{6} \right) \] (2.30)

where:

- \( E_p \) = elastic modulus of the prestressing strands
- \( X_h \) = distance from harp point to center of span
- \( L_{db} \) = average unbonded length of the unbonded strands
- \( L_t \) = transfer length of prestressing strands
$e_m = \text{eccentricity of the centroid of the strands at mid-span with respect to the centroid of the gross section}$

$$e_m = y_c - y_{psm} \quad (2.31)$$

$y_{psm} = \text{distance from centroid of prestressing steel to extreme bottom fibers of the section calculated at mid span}$

$e_e = \text{eccentricity of the centroid of the strands at the end of the girder with respect to the centroid of the gross section. unbonding is neglected.}$

$$e_e = y_c - y_{pse} \quad (2.32)$$

$y_{pse} = \text{distance from centroid of prestressing steel to extreme bottom fibers of the section, calculated at the end of the member}$

The deflection due to live load of girder $\Delta_{LL}$ is related to the HL-93 truck position on simple spans, $x_1$ as shown on Figure 2.8 and can expressed in equation (2.33) (Praveen, 2009 and Cory, 2014).

$$x_1 = \frac{473}{650+9.31} \quad (2.33)$$

where:

$l = \text{span length (m)}$
Table 2.7 Beam deflection due to concentrated load at any point

<table>
<thead>
<tr>
<th>Beam type</th>
<th>Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam simply supported at End – Concentrated load P at any point</td>
<td>$y = \frac{Pbx}{6EI} (l^2 - x^2 - b^2)$ for $0 &lt; x &lt; a$</td>
</tr>
<tr>
<td></td>
<td>$y = \frac{Pbx}{6EI} \left( \frac{l}{b} (x - a)^2 (l^2 - b^2) x - x^3 \right)$ for $a &lt; x &lt; l$</td>
</tr>
</tbody>
</table>

Source: Gere (2004)

while the deflection due to live load $\Delta_{LL}$ can be estimated on deflection of beam theory from concentrated load at any point that related to the position of HL-93 truck’s wheel was taken in table 2.7 (Gere, 2004). Criteria for deflection are specified on the article 2.5.2.6.2 of the AASHTO (2007). The deflection limitation shall be considered for concrete construction in the following:

- General vehicular load with Span/800
- Vehicular and/or pedestrian load with Span/1000
- Vehicular load on cantilever arms with Span/300

Vehicular and/or pedestrian load on cantilever arms with Span/375

2.5 Deterioration prediction model

As discussion of article 2.2 about deterioration of steel corrosion, it can divided corrosion process into an initiation and a propagation phases. For this prediction models will be determined conform to the corrosion process. The first is the initial stage that concerning of deterioration of carbonation and chloride ingress models. The second is the propagation phase which is studying of crack initiation, the first warning sign of this stage and the amount of steel section loss that has achieved significant reduction of the performance of structures.

32
2.5.1 Carbonation

It has been commonly accepted that carbonation depth is proportional to square root of carbonation time. According to DPT 1332-2007 presented that when the concrete is in the environment faced by carbon dioxide. Carbon dioxide is released into concrete and reacting carbonation, which makes the ability of concrete to prevent corrosion of steel reinforcement decreased. So carbonation can present by equation (2.34).

\[
C \geq \gamma_i X_c
\]  
\hspace{1cm} (2.34)

where:
- \( C \) = covering depth (mm)
- \( \gamma_i \) = 1.0 for structures that require a lifetime of non-maintenance less than 15 years.
  1.1 for structures that require a lifetime of non-maintenance more than 15 years.
- \( X_c \) = carbonation depth at considering time (mm)

in which:

\[
X_c = \alpha_1 \alpha_2 k \sqrt{t_r}
\]  
\hspace{1cm} (2.35)

where:
- \( t_r \) = lifetime of concrete (years)
- \( \alpha_1 \) = 1.0 for dry surface concrete and 0.95 for wet surface concrete
- \( \alpha_2 \) = coefficient of environmental carbonation, table 2.8
- \( k \) = coefficient of carbonation (mm/√years)

<table>
<thead>
<tr>
<th>Violence of environmental</th>
<th>( \alpha_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal situation</td>
<td>0.65</td>
</tr>
<tr>
<td>Medium violence of carbonation</td>
<td>0.85</td>
</tr>
<tr>
<td>High violence of carbonation</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Source: DPT 1332-2007
2.5.2 Chloride

According to DPT 1332-2007 reported that when the concrete is in the environment reached by chloride. The deterioration of concrete due to chloride attack involves the diffusion of chloride. If chloride concentration at steel has more than chloride critical, it will destroy the passive protective film on steel which prevents corrosion of steel reinforcement that can express by equation (2.36).

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

(2.36)

where:
- \( Cl_d \) = chloride concentration (percent by weight of binder)
- \( Cl_s \) = chloride concentration at surface of concrete (kg/m\(^3\))
- \( C \) = covering depth (mm)
- \( B \) = weight of binder (kg/m\(^3\))
- \( Da \) = Chloride Diffusion (cm\(^2\)/year)

2.5.3 Corrosion induced concrete cracking

As the corrosion induced concrete cracking is the first warning sign of propagation stage in corrosion. Leon and Dimitri (2011) have been summarized the critical formulation of time to corrosion induced concrete cracking into empirical, analytical and numerical models for prediction. The time to first cracking, \( t_{cr} \) estimated using empirical models which is relating to the amount of corrosion product, \( W'_{\text{steel,crr}} \) and corrosion current density in steel, \( i_{\text{corr}} \) can expressed on equation (2.37).

\[
t_{cr} = \frac{W'_{\text{steel,crr}}}{0.009113 \times i_{\text{corr}}}
\]

(2.37)

where, \( t_{cr} \) is time of corrosion induced concrete cracking in years, \( W'_{\text{steel,crr}} \) is the critical mass loss of steel that equal to 0.01 g/cm\(^2\) (JSCE. 2007) and \( i_{\text{corr}} \) is the current density in \( \mu A/cm^2 \).
2.5.4 Corrosion induced spalling of concrete cover

Some researches (VU and Stewart. 2002 and Christensen. 2010) have been proposed time to spalling model due to amount of rust product can be determined as equation (2.38). In additional, area of spalling of concrete can be estimated by using the Monte Carlo simulation.

\[ T_{sp} = [0.84 (t_{ser} + t_{1st} + 0.2)]^{1.4} \]  
\[ (2.38) \]

where \( T_{sp} \) is time of corrosion induced concrete spalling in years, \( t_{ser} \) is time to severe cracking and \( t_{1st} \) is time to initiation cracking.

in which

\[ t_{ser} = [A \times 10^{-3} \times (wc/C)^{-B}] \times \frac{100}{i_{corr}} \]  
\[ (2.38) \]

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.

2.5.5 Steel loss prediction

Many researches (Maaddawy et al.2003, Maaddawy et al. 2007, Mohamed et al.2010 and Chunhua et al. 2011) have used Faraday’s law to calculated the weight loss of steel from applied current density, and its ability to predict the actual loss of steel at different current density. It can be calculated using the following expression based on Faraday’s law as shown in equation (2.40).

\[ M_{loss} = \frac{MIT}{ZF} \]  
\[ (2.40) \]

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).

To determine the relationship between the percentage steel mass loss, \( m_l \) and time, Maaddawy et al. (2007) has been summarized the formula that can be expressed as equation (2.41).
where $T$ is the time (days), $m_1$ is the percentage of steel mass loss, $D$ is diameter of the steel reinforcing bar (mm).

### 2.6 A probabilistic approach for modeling deterioration

Investigation of structure to evaluate the performance of structure, inadequate information can make the accuracy of deterioration prediction is inevitably low (JSCE, 2007) as shown in Figure 2.9.

There are several parameters to investigate such as quality of concrete and steel, and exposure conditions, to develop the accuracy prediction of structure, Sanjeev et al. (2014) reported that probabilistic approach is appropriately improve in the deterioration prediction. There are a number of modeling techniques for predicting based on probability concepts such as Markov chain analysis, probability distribution, Monte Carlo simulation and etc. There are some of researches (Tesfamariam and Martín-Pérez 2008 and Ferreira 2004) 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.

![Figure 2.9 Accuracy of deterioration prediction](image-url)
2.6.1 Monte Carlo simulation

Monte Carlo simulation is a kind of probabilistic analysis that depends on replicated random number of experiments and statistical analysis of the experiments to compute the conclusions on model output as described on Figure 2.10 (Raychaudhuri. 2008 and Xiaoping. 2005). In this simulation can be distinguished into three parts. First, Generating samples of random parameter in the interval of [0,1] to transform their variables in the suitable distributions that have well known in cumulative density functions (cdfs) for input the simulation experiments. Second, using of their mean, standard deviations and etc. from distributions value to simulated experiments by a large number of samples has significant results clearly in term of this probabilistic analysis theory. Last, statistical analysis of experiment results has determined to conclusion the experiment. Generally, the statistical of probabilistic analysis has represented in kind of reliability, the probability of failure, probability density functions (pdfs) and cumulative density functions (cdfs) for easily understood the behavior of their problems by non-mathematicians.

The probability of failure \( p_f \) can be estimated by the following:

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

(2.42)

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 standard error of the probability of failure (\( s \)) can be estimated by:

\[
s = \sqrt{\frac{p_f (1-p_f)}{N}}
\]

(2.43)
Step 1: Sampling of random variables
Generating samples of random variables

Step 2: Numerical Experimentation
Evaluating performance function

Analysis Model
\( Y = g(X) \)

Step 3: Statistical Analysis on model output
Extracting probabilistic information

Distributions of input variables

Samples of input variables

Samples of output variables

Probabilistic characteristic of output variables

Figure 2.10 Monte Carlo simulation
Chapter 3
Methodology

3.1 General

This chapter was explained software to predict performance of existed prestressed concrete bridge girders. Conceptual framework was limited for this study, the mathematical implementation for deterioration prediction of prestressed concrete bridge girder will be proposed and developing the software based on proposed model. All of topics were presented in the following sections.

3.2 Conceptual framework

Program was developed for calculating deterioration prediction of prestressed concrete bridge girder in term of probability of failures for supporting the maintenance engineer. There are three main stages of the process consisting of input the parameters for simulation, computed the structural performance with different criteria and summarized the probability of failure that can be illustrated in Figure 3.1. First, the input variables required in this computerized program are material properties, structure dimension and environment condition. Second, predicting the deterioration and performance of structure were conducted during service life. In this study, three types of limit states are concerned.

- Durability limit states are related to steel corrosion from corrosion initiation until loss of steel section that effect directly to other limit states such as crack initiation, prestressing force for serviceability state and load carrying for ultimate state.
- Serviceability limit states are considered to allowable stress of steel, tendon and concrete and deflection.
- Ultimate limit states are mostly related to load carrying of structure that comprise of flexural strength and shear strength
Finally, the result of statistical analysis in term of probability of failure is concluded. The overall conceptual framework of this study is calculated based on Monte Carlo simulation.
3.3 Defining model parameters

The various parameters are taken from the assessment of the structure. In order to evaluate the performance of structure, defining model parameters was required. There are two kinds of data that need for taking into model proposed:

- **Technical specification data** is related to information such as the general information, dimension of structure and properties of materials that normally obtained from the contract drawing of their structures. The general information is presented about the project name, route and location of structure and construction years. For dimension of structure is introduced by type of girder, girder length and spacing of girder.

In case of properties of materials is comprised of: Properties of concrete such as compressive strength of concrete ($f_{c'}$), covering of concrete (c) and mix proportion of concrete, Properties of prestressing strands (PS) such as diameter of PS, number of PS, tensile strength of PS ($f_{pu}$) and modulus of elastic of PS ($E_p$), Properties of reinforced steel such as diameter of steel, yield strength of reinforced steel ($F_y$) and modulus of elastic of reinforced steel ($E_s$).

- **Inspection data** is involved with the information that assess directly on the structure such as compressive strength of concrete ($f_{c'}$), covering of concrete (c), carbonation depth ($X_c$) and corrosion rate ($i_{corr}$). Even as environmental condition such as relative humidity (RH), temperature and carbon dioxide concentration were directly measured, while chloride concentration at surface of concrete ($Cl_s$) and chloride diffusion ($D_a$) can obtained from laboratory test on structural samples.
3.4 Proposal for deterioration prediction model

In this study designing, deterioration prediction of prestressed concrete bridge girder were separated to three limit states that comprise with durability, serviceability and ultimate limit states. The proposed model is based on the background as literature, but only necessary alterations are made to the model.

3.4.1 Durability limit states

The durability characteristic of concrete for service life of structure is depending on their environment. One of global problem is corrosion of metal in concrete. The process of corrosion are initially destroyed by either carbonation or chloride, and inducing concrete to crack and spall respectively.

3.4.1.1 Carbonation

The natural alkalinity of the surface concrete are reduced by acidic gases (carbon dioxide) which results in corrosion when it reaches the steel. The deterioration estimation initially calculated the initiation corrosion. According to DPT 1332-2007 presented the carbonation by equation. (3.1).

\[ C \geq \gamma_i X_c \]  

(3.1)

where \( C \) is covering depth (mm), \( \gamma_i \) are 1.0 for structures that require a lifetime of non-maintenance less than 15 years and 1.1 for structures that require a lifetime of non-maintenance more than 15 years and \( X_c \) is carbonation depth at considering time (mm). By solving this carbonation depth, equation (3.2) is obtained

\[ X_c = \alpha_1 \alpha_2 k \sqrt{t_r} \]  

(3.2)

where \( t_r \) is lifetime of concrete (years), \( \alpha_1 \) is 1.0 for dry surface concrete and 0.95 for wet surface concrete, \( \alpha_2 \) is coefficient of environmental carbonation (see table 2.7) and k is coefficient of carbonation (mm/√years).
3.4.1.2 Chloride

The ingress of chloride ions (Cl\textsuperscript{-}) is attacked into the surface concrete. Corrosion initiated when Cl\textsuperscript{-} concentration exceeds 0.4% by weight of binder (Cl\textsubscript{lim}) that could be taken from DPT 1332-2007 by using equation. (3.3).

\[ Cl_d < Cl_{lim} \]  

(3.3)

in which

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

(3.4)

where \( Cl_d \) is chloride concentration (weight of binder), \( Cl_s \) is chloride concentration at surface of concrete (kg/m\textsuperscript{3}), \( C \) is covering depth (mm), \( B \) is weight of binder (kg/m\textsuperscript{3}) and \( D_a \) is chloride diffusion (cm\textsuperscript{2}/year).

3.4.1.3 Loss of steel section

The propagation stage in corrosion, the passive oxide film has been destroyed. Starting of steel corrosion in concrete from this stage can occur. Loss of steel section became to be serious problem to other limit states that can be obtained based on Faraday’s law by using equation. (3.5).

\[ m_1 = \frac{T \times i_{corr}}{78.3D} \]  

(3.5)

where \( T \) is the time (days), \( m_1 \) is the percentage of steel mass loss, \( D \) is diameter of the steel reinforcing bar (mm) and \( i_{corr} \) is the current density in (\( \mu A/cm^2 \)).

3.4.1.3 Corrosion induced concrete cracking

The time to first cracking, \( t_{cr} \) estimated using empirical models Leon and Dimitri (2011) which is relating to the amount of corrosion product, \( W'_{steel,cr} \) and
corrosion current density in steel, \( i_{\text{corr}} \) was estimated based on Faraday’s law by using equation (3.6).

\[
t_{\text{cr}} = \frac{W'_{\text{steel,cr}}}{0.009113 \cdot i_{\text{corr}}}
\]  

(3.6)

where, \( t_{\text{cr}} \) is time of corrosion induced concrete cracking in years, \( W'_{\text{steel,cr}} \) is the critical mass loss of steel that equal to 0.01 g/cm\(^2\) (JSCE, 2007) and \( i_{\text{corr}} \) is the current density in \( \mu\text{A/cm}^2 \).

### 3.4.1.4 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 \( (T_{\text{sp}}) \) determined by using empirical models (VU and Stewart. 2002 and Christensen. 2010) as shown in equation (3.7).

\[
T_{\text{sp}} = [0.84 \cdot (t_{\text{ser}} + t_{\text{1st}} + 0.2)]^{1.4}
\]

(3.7)

where \( T_{\text{sp}} \) is time of corrosion induced concrete spalling in years, \( t_{\text{ser}} \) is time to severe cracking and \( t_{\text{1st}} \) is time to initiation cracking.

in which

\[
t_{\text{ser}} = [A \times 10^{-3} \times (w_c/C)^{-B}] \times \frac{100}{i_{\text{corr}}}
\]

(3.8)

where \( w_c \) 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.

### 3.4.2 Serviceability limit states

Actions to be concerned at the serviceability limit state shall be stress limitations for concrete and prestressing tendons and deflection. After corrosion, loss of cross section of prestressing tendons is probably leading to yielding and fracture which affect directly to reduce deflection of members.
3.4.2.1 Stress limitations for prestressing tendons

As a result of reduction in prestressing tendons sectional area due to corrosion has affected to increasing of stress. Consequently, the allowable stress of steel AASHTO (2007) form steel section loss shall not exceed specifications in table 3.1

<table>
<thead>
<tr>
<th>Tendon Type</th>
<th>Stress-Relieved Strand and Plain High – Strength Bars</th>
<th>Low Relaxation Strand</th>
<th>Deformed High – Strength Bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Condition</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pre-tensioning</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Immediately prior to transfer</td>
<td>0.70 $f_{pu}$</td>
<td>0.75 $f_{pu}$</td>
<td>-</td>
</tr>
<tr>
<td>At service limit state after all losses</td>
<td>0.80 $f_{pu}$</td>
<td>0.80 $f_{pu}$</td>
<td>0.80 $f_{py}$</td>
</tr>
</tbody>
</table>

3.4.2.2 Stress limits for concrete

Allowable stress of concrete, the total stress that consists of prestressing force and moment capacity as shown in equation (3.9) shall not exceed the specifications AASHTO (2007) in section the following.

a) extreme fiber stress in compression due to prestress including sustained loads for $0.45 f'_c$
b) extreme fiber stress in compression due to prestress including total loads for $0.60 f'_c$
c) extreme fiber stress in tension due to prestressed tensile zone for $0.5 \sqrt{f'_c}$

$$f_c = \frac{-P}{A_g} \pm \frac{P_{ey}}{I_g} \mp \frac{My}{I_g}$$

(3.9)
where $f_c$ is concrete stress at the $y$ (MPa), $P$ is prestressing forced (MN), $A_g$ is gross area ($m^2$), $e$ is eccentricity of strands with respect to centroid of girder (m), $y$ is distance from the neutral axis to extreme fiber (m), $I_g$ is gross moment of inertia ($m^4$) and $M$ is flexural moment (MN-m) in which (+) is tension concrete and (-) is compression concrete

### 3.4.2.3 Deflection

The total deflection, combination of deflection due to prestressing, self-weight and live load was calculated from equation (3.10).

$$\Delta_t = \Delta_{ps,t} - \Delta_{sw,t} - \Delta_{LL}$$  \hspace{1cm} (3.10)

where:

- $\Delta_t$ = total deflection of girder (mm)
- $\Delta_{ps,t}$ = deflection due to prestressing girder (mm)
- $\Delta_{sw,t}$ = deflection due to girder self-weight (mm)
- $\Delta_{LL}$ = deflection due to live load of girder (mm)

The deflection due to self-weight of girder $\Delta_{sw,t}$ is given in equation (3.11).

$$\Delta_{sw,t} = \frac{5w_g L^4}{384E_{ci}I_g}$$  \hspace{1cm} (3.11)

where:

- $w_g$ = uniformly distributed girder self-weight
- $L$ = girder length
- $I_g$ = moment of inertia of the girder cross section

The deflection due to prestressing girder $\Delta_{ps,t}$ is taken as equation (3.12).

$$\Delta_{ps,t} = \Delta_{ps,i} - \frac{P_i - P_t}{E_{ci} + E_c} \left( \frac{e_m L^2}{8} - \frac{(e_m - e_e) (L/2 - x_h)^2}{6} - \frac{e_m (L_{db} + L_t)^2}{6} \right)$$  \hspace{1cm} (3.12)

in which:
\[ \Delta_{ps,i} = \frac{P_i}{E_{ci} l_g} \left( \frac{e_m L^2}{8} - \frac{(e_m - e_e)(L/2 - X_h)^2}{6} - \frac{e_m(L_{db} + L_t)^2}{6} \right) \] (3.13)

where:

- \( E_p \) = elastic modulus of the prestressing strands
- \( X_h \) = distance from harp point to center of span
- \( L_{db} \) = average unbonded length of the unbonded strands
- \( L_t \) = transfer length of prestressing strands
- \( e_m \) = eccentricity of the centroid of the strands at mid-span with respect to the centroid of the gross section
  \[ = y_c - y_{psm} \] (3.14)
- \( y_{psm} \) = distance from centroid of prestressing steel to extreme bottom fibers of the section calculated at mid span
- \( e_e \) = eccentricity of the centroid of the strands at the end of the girder with respect to the centroid of the gross section. unbonding is neglected.
  \[ = y_c - y_{pse} \] (3.15)
- \( y_{pse} \) = distance from centroid of prestressing steel to extreme bottom fibers of the section, calculated at the end of the member

In case of deflection due to live load of girder (\( \Delta_{LL} \)), can be determined by deflection of beam theory from concentrated load at any point that related to the position of HL-93 truck’s wheel was calculated in table 3.2. In additional, the maximum force of HL-93 truck on girder can be calculated by the distance of second wheel (\( x_i \)) from mid of span. It has been shown in equation (3.16) which l is span length in meter. The criteria for total deflection of this study shall not exceed to span/800 for general vehicular load

\[ x_i = \frac{473}{650 + 9.31} \] (3.16)
Table 3.2 Beam deflection due to concentrated load at any point

<table>
<thead>
<tr>
<th>Beam type</th>
<th>Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam simply supported at End</td>
<td>$\Delta_{LL} = \frac{Pbx}{6EI} (l^2 - x^2 - b^2)$ for $0 &lt; x &lt; a$</td>
</tr>
<tr>
<td></td>
<td>$\Delta_{LL} = \frac{Pbx}{6EI} l b \left( x - a \right)^3 \left( l^2 - b^2 \right) x - x^3$ for $a &lt; x &lt; l$</td>
</tr>
</tbody>
</table>

Source: Gere (2004)

Later, loss of prestress can be computed from equation (3.19) by refined estimates of time dependent losses. For each stage of prestress loss, elastic shortening, shrink loss, creep and relaxation were determined respectively.

$$\Delta f_{pTL} = \Delta f_{pES} + (\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR_l})_d + (\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR_2} - \Delta f_{pSS})_d$$ (3.19)

where:

$(\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR_l})_d$ = sum of time dependent prestress losses between transfer and deck replacement (MPa)

$(\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR_2} - \Delta f_{pSS})_d$ = sum of time dependent prestress losses after deck replacement (MPa)

The loss of prestress prediction procedure for proposed refined method is as follows:

1) Determine the elastic shortening loss according to the AASHTO 2007 procedure provided in Section 2.4.8.1.

2) Determine the time-dependent losses of shrinkage according to the AASHTO 2007 and shrinkage strain is determined according to DPT 2007 procedure provided in Section 2.4.8.2.
3) Determine the time-dependent losses of creep according to the AASHTO 2007 procedure provided in Section 2.4.8.3.

4) Determine the time-dependent losses of relaxation of prestressing strands according to the AASHTO 2007 procedure provided in Section 2.4.8.4.

5) Determine the time-dependent losses of shrinkage of deck after transfer according to the AASHTO 2007 procedure provided in Section 2.4.8.5.

3.4.3 Ultimate limit states

Both of flexural and shear strength capacity was calculated from section as shown in Figure 3.2 and material properties and multiple with strength reduction factor on table 2.4. 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 of load.

3.4.3.1 Flexural strength

Flexural of reinforcement concrete members must be designed to resist the force at ultimate states. Responding of structure at ultimate was analyzed to make sure that structure will not collapse suddenly under service load as shown in equation (3.20).

\[
M_n = A_{ps} f_{pu} (d_p - a/2) + A_s f_s (d_s - a/2) - A'_{s} f'_s (d'_s - a/2) + 0.85 f'_c (b - b_w) h_f (a/2 - h_f/2)
\]

(3.20)

where \( A_{ps} \) is area of prestressing steel (mm\(^2\)), \( f_{pu} \) is stress in prestressing steel at nominal (MPa), \( d_p \) is effective depth to prestressing tendons (mm), \( A_s \) is area of nonprestressed tension reinforcement (mm\(^2\)), \( f_s \) is stress in mind steel tension reinforcement at nominal (MPa), \( d_s \) is effective depth to mind steel tension reinforcement.
reinforcement (mm), $A'_s$ is area of compression reinforcement ($mm^2$), $f'_s$ is stress in mind steel compressive reinforcement at nominal (MPa), $d'_s$ is effective depth to compression reinforcement (mm), $f'_c$ is specified compressive strength of concrete at 28 days (MPa), $b$ is width of compression face of the member (mm), $b_w$ is web width of section (mm), $h_f$ is compression flange depth of member (mm), $a$ is depth of the equivalent stress block (mm)

Figure 3.2 Cross section of I girder
3.4.3.2 Shear strength

Design of prestressed concrete member to against shear resulting from externally service load which failure is different from flexural behavior shall be taken as equation (3.21).

\[ V_n = V_c + V_s + V_p \]  
\[ (3.21) \]

in which:

\[ V_c = (0.05 \sqrt{f'_c} + 4.8 \frac{V_u d_p}{M_u}) b_w d \]  
\[ (3.22) \]

\[ \text{by } \frac{V_u d_p}{M_u} \leq 1 \text{ and } 0.17 \lambda \sqrt{f'_c} b_w d_p \leq V_c \leq 0.42 \lambda \sqrt{f'_c} b_w d_p \]

\[ V_s = \frac{A_v f_y d_v \cot \theta}{s} \]  
\[ (3.23) \]

\[ V_p = P \sin \theta \]  
\[ (3.24) \]

where \( d \) is depth of c.g. of all tensile reinforcements > 0.8h (mm), \( \lambda \) is 1.00 for normal weight concrete, 0.85 for sand light-weight concrete and 0.75 for all light-weight concrete, \( A_v \) is area of shear reinforcement within a distances (mm\(^2\)), \( d_v \) is effective shear depth (mm), \( s \) is spacing of stirrups (mm) and \( P \) is prestressing force (N)

3.5 Probabilistic approach

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. In the present software, 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 the following:

\[ p_f = \frac{1}{N} \cdot \sum_{j=1}^{N} 1[\text{g}(X_j)] \]  
\[ (3.25) \]
where \( N \) is the number of simulation, \( I[g(X_j)] \) is the indicator function and \( X_j \) is the \( j \)th sample drown according to the probability density function \( f(x) \).

3.6 Assumption and limitations

The users need to understand several assumptions concerning the design used in the evaluation the performance of structure. Therefore, the necessary information are assume in the following:

- Only pre-tensioned system of prestressed concrete (I-girder) is concerned which is different from post tensioned system.
- The progress of steel corrosion is constant damage after cracking due to corrosion because the rate of steel corrosion after cracking is affected by the quantities of water and oxygen that prediction is difficult.
- Loss of concrete is neglected, effect on stiffness of section that these standard equations of prediction have not been solved
- Capacity designs are concerned at mid-span for flexural strength and at end span for shear strength.
- Bridge design loading was considered at dead load and live load in strength I of load combination limit state according to AASHTO 2007.
- Bridge structure has many girder per lane, so distribution of live load to girder is considered according to AASHTO 2007
- Only normal distribution of variables was considered in statistical analysis software.
- Shear failure was considered on girder only and excluded from slab which can resist in shear.

3.7 Development of software based on the proposed model

The developed software was given the explanation in this section. The software is written in visual basic and has a visual interface written in visual basic also. The software represents the application of the model proposed in section 3.3 which developed to Monte Carlo simulation.

Figure 3.3 illustrates the introduction and main window of the software. The software is comprised of three different options:
1) Data for input the variables
2) Perform a simulation for predicting the performance of structure on their criteria.
3) Export data to Microsoft excel for convenient analysis

Figure 3.3 Introduction window

3.7.1 Input data

In the input option, the general information is introduced along with the model parameters. This optional window is consist of three kinds of selection: create an input information for generating the data files to simulation, edit an existing information for revising the previous data files and view former input information to observe any existing data files.

In general information is comprised of the project name, route, location of beam in term of station and global positioning system (GPS), construction years, date on generated data files and the important information of simulation like time to prediction and number of iteration for Monte Carlo simulation. The model parameters are separated into seven sections: section of concrete, prestressing strand, reinforcing steels, material properties, environmental surrounds on their structure, results of inspection and other section such as mix proportion of concrete.
Figure 3.4 Section of concrete window

The section of concrete window, the dimension of I girder is introduced by type of girder as well as girder length and spacing of girder as shown Figure 3.4. The prestressing strand window is where the user defines the information about number of strand, covering of concrete, strand size, strand type as well as the distance of strand from centroid as shown Figure 3.5.

Figure 3.5 Prestressing strand window
The reinforcing steel window is where the user identifies the data related to the characteristic of reinforcing steel and covering of concrete as shown Figure 3.6. All material properties shall key on material properties window as shown Figure 3.7.
Figure 3.8 Environmental condition

The environmental condition window, relative humidity, temperature, rain subjection and carbon dioxide concentration are presented as shown Figure 3.8. Carbonation and chloride test, corrosion rate, strength and covering of concrete for inspection results window on Figure 3.9. Mix proportion is represented on other window as shown Figure 3.10.

Figure 3.9 Results of inspection
3.7.2 Simulation

There are two kinds of deterioration prediction which comprised with average years failure prediction and probability of failure according to Monte Carlo simulation as shown on Figure 3.11. In average value is determined for time to failure due to their criteria in term of years as presented on Figure 3.12.
Figure 3.12 Deterioration prediction in average value window

In probability of failure, the software performs the model simulation according to Monte Carlo simulation which considerable depending on many factors such as number of iteration, number of time prediction, among other. The software displays the results in graphs on durability, serviceability and ultimate. Figure 3.13 is an illustration of graphs presented.

Figure 3.13 Probability of failure window
3.7.3 Export data

Once the simulation has ended, the results are always displayed in the pattern of a single graph only in software. In order to take a better convenient to analyze the simulation output for comparison of results from various analyses, it can be converting the results from software into MS Excel as presented on Figure. 3.14.

![Figure 3.14 Export data to Microsoft Excel](image)

3.7.4 Database

In order to accumulate their information for displaying on the software, it used Microsoft access to manage the data. In database comprise with three parts. First,
input information of their project records which including of general information, technical specification data such as section and properties of materials and inspection data such as environment condition and inspection results as shown in Figure. 3.15.

Second, the results of simulation as average years failure prediction as shown in Figure. 3.16. Third, probability of failure database is shown in Figure. 3.17. In addition, this database can applied for ranking of maintenance decisions by filter out of information that need for consideration.
Figure 3.15 Input information records in database

Figure 3.16 Average years failure prediction records in database
Figure 3.17 Probability of failure records in database
3.8 Case studies

The bridge considered in this study is a simple span prestressed concrete bridge in Bangkok was constructed in 1981, has span length 20 m, 0.18 m effective slab thickness, 1.975 m effective width and dimension of girder as shown in Figure 3.18. 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).

![Figure 3.18 Dimension of prestressed concrete girder (mm.)](image)

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 3.3 shows the parameters which can get from inspection have some uncertainty value. The coefficient of variation of these parameters ranges.
Table 3.3 Parameters of prediction

<table>
<thead>
<tr>
<th>Parameter</th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
<th>G4</th>
<th>G5</th>
<th>G6</th>
<th>G7</th>
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<td>COV</td>
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<td>0.16</td>
<td>0.16</td>
<td>0.16</td>
<td>0.16</td>
<td>0.16</td>
<td>0.16</td>
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</tr>
<tr>
<td>$f_{py}$, Yield strength (MPa)</td>
<td>1725</td>
<td>1725</td>
<td>1725</td>
<td>1725</td>
<td>1725</td>
<td>1725</td>
<td>1725</td>
<td>Mahmoodian (2012)</td>
</tr>
<tr>
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<td>0.28</td>
<td>0.28</td>
<td>0.28</td>
<td>0.28</td>
<td>0.28</td>
<td>0.28</td>
<td></td>
</tr>
<tr>
<td>Surface chloride content (% of binder)</td>
<td>2.95</td>
<td>2.95</td>
<td>2.95</td>
<td>2.95</td>
<td>2.95</td>
<td>2.95</td>
<td>2.95</td>
<td>Darmawan &amp; Stewart (2007)</td>
</tr>
<tr>
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<td>0.1</td>
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<td>0.1</td>
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<td></td>
</tr>
<tr>
<td>Chloride diffusion (cm$^2$/years)</td>
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<td>0.4</td>
<td>0.5</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
<td>Darmawan &amp; Stewart (2007)</td>
</tr>
<tr>
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<td>0.2</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>$i_{corr}$, Corrosion rate ($\mu$A/cm$^2$)</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>Darmawan &amp; Stewart (2007)</td>
</tr>
<tr>
<td>COV</td>
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<td>Number of tendon</td>
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<td>26</td>
<td>26</td>
<td>26</td>
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<td>22</td>
<td>Figure 3.19</td>
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</table>

Figure 3.19 Different number of prestressing strand
Chapter 4
Results and discussions

4.1 General

This chapter presented the results and discussions of case study simulations, which were assumed to different type of condition. The deterioration prediction and the effect of probability of failure are presented in section 4.2 and 4.3 respectively. In addition, section 4.4 presents results and discussions on the priority of maintenance decision for maintenance planning. The results and discussions are presented in the following sections.

4.2 Deterioration prediction analysis after corrosion

The predictive method was used to determine the need for maintenance of prestressed concrete bridge girder, the criterion of deterioration prediction need to understand for relative results especially after corrosion process. However, It is well known that durability limit states of failure is related to corrosion but for serviceability and ultimate limit states are related to the capacity of structure as results of corrosion like section loss of steels or prestressing strands. Serviceability limit states, loss cross section of prestressing steel cause strand stress to increase.

![Relaxation of prestressing strands](image)

Figure 4.1 Relaxation of prestressing strands
Consequently, relaxation is increased which is calculated from equation (2.24) as shown on Figure 4.1. Figure 4.2 shows the effect of deflection due to section loss of prestressing that decrease to camber due to prestressing as determined on equation (3.12). Similar to the results of total deflection as shown on Figure 4.3 that reduce the camber of prestressed concrete girder but it still in the limitation of deflection criterion because of not only the prestressing force might be affected, the stiffness of concrete is also one of criteria due to deflection as calculated from equation (3.10).

![Deflection due to prestressing](image1)

**Figure 4.2 Deflection due to prestressing**

![Total deflection](image2)

**Figure 4.3 Total deflection including of prestressing, self-weight and live load**

In case of ultimate limit states, the capacity of structure will be reduced according to the reduction of steel or prestressing section as seen in Figure 4.4 and 4.5.
However, it is different mechanisms of steel and prestressing corrosion. For reinforcement steel will be reduced the capacity related to loss of section but in case of prestressing strand will be failed suddenly when exceeding of allowable of strand that conform to the results of strand failure of prestressed concrete bridge girder Darmawan et al. (2007).

![Flexural capacity](image1)

![Shear capacity](image2)

**4.3 Probability of failure analysis**

In order to demonstrate how the program can be applied, some of typical results of probability of failure and discussions are given in the following. Basically, three types of limit states are concerned that consist of durability, serviceability and ultimate limit states.
4.3.1 Effect of the number of iteration

For illustrative purpose in this study, the replicated of random number of simulation will be used to determine the effect of iteration has on probability of failure analysis due to prestressed concrete bridge girder as shown in Figure 4.6 to Figure 4.8. Experimental comparisons of serviceability limit states results of different number of iteration with 500, 1000 and 2000 respectively.

![Figure 4.6 Probability of failure based on allowable stress of concrete and prestressing steel at iteration 500 times](image)

![Figure 4.7 Probability of failure based on allowable stress of concrete and prestressing steel at iteration 1000 times](image)
The results showed that for number of iteration of 500 has rough tendency of probability of failure. In case results of number of iteration in 1000 and 2000 had smooth tendency than 500 repetitions. So, it can be concluded that the different number of iteration affect only the characteristic of tendency. If the high number of random the results will be more accuracy for time dependent of failure analysis.

Figure 4.8 Probability of failure based on allowable stress of concrete and prestressing steel at iteration 2000 times

4.3.2 Case studies analysis

Based on the data as section 3.7, the analysis shows the effect on time dependent probability of failure such as different environment condition, covering of concrete and number of strand. The reliability of prestressed concrete bridge girder was evaluated on each performance criterion which the statistic parameters are shown in Table 3.3.

4.3.2.1 Probabilistic analysis of the durability limit state criteria

4.3.2.1.1 Effect of environment

Corrosion initiation, crack and spalling of concrete results can be seen in Figure 4.9 (a) and (b) which show that aggressive environment tends to had higher probability of failure. For corrosion initiation, increase chloride diffusion from 0.3 to 0.5 (cm²/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 (µA/cm²), it can lead to decrease the service life about 2.5
to 5 years for cracking but for spalling of concrete can lead to reduce the performance about 20 to 25 years.

Figure 4.9 Effect of environment on durability limit state

4.3.2.1.2 Effect of concrete cover

Figure 4.10 (a) and (b) show results of probability of failure due to durability. It shows that less of concrete cover tends to had higher probability of failure. For corrosion initiation, reduce covering of concrete from 25 to 15 (mm) results in increase of probability of failure from about 5 to 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 time to failure in cracking increase from about 2.5 to 5 years. Similarly results of spalling of concrete that increase from 15 to 20 years.

**Figure 4.10 Effect of concrete cover on durability limit state**

a) Corrosion initiation criteria

b) Crack initiation and spalling of concrete criteria

4.3.3.2 Probabilistic analysis of the serviceability limit state criteria

4.3.3.2.1 Effect of environment
Figure 4.11 presented probability of failure due to serviceability limit states. It was found that there results had similar tendencies with the results of durability limit states which 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 had higher probability of failure.

![Figure 4.11: Allowable stress of concrete and prestressing steel criteria due to environment condition](image)

![Figure 4.12: Allowable stress of concrete and prestressing steel criteria due to covering of concrete](image)

4.3.3.2.2 Effect of concrete cover
The results of probability of failure due to serviceability limit states are discussed in Figure 4.12. It can be seen that time to failure of both of allowable stress in concrete and prestressing steels are increase to 10 years due to reducing of 5 mm of covering of concrete. Especially, G5 is the most concerned because thickness of concrete cover is smaller than other.

4.3.3.3 Probabilistic analysis of the ultimate limit state criteria

4.3.3.3.1 Effect of environment

Figure 4.13 presented probability of failure due to ultimate limit states. 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.

![Figure 4.13 Effect of environment on ultimate limit state](image)

4.3.3.3.2 Effect of concrete cover

The results of probability of failure due to ultimate limit states are discussed in Figure 4.14. 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.

![Graph showing probability of failure over time](image)
4.3.3.3 Effect of number of strand failure

Figure 4.15 show 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 time to failure for prestressing failure only. As can be seen the graph, the mean time to failure reduce from 60 years to 50 years if the number of strands decrease by 2 strands.

In summary, the aggressive environment condition indicated that tends to more severely encounter the problem of probability of failure of prestressed concrete bridge.
girder. This is agreement with Darmawan et al. (2007) and Mahmoodian et al. (2012) that the aggressive environment is the most important single factor that affects the probability of failure.

In addition, effect of different covering of concrete reported that less of concrete cover leads to increase the time to failure of prestressed concrete bridge girder. Similar to Ferreira (2004) that demonstrate the reduction of covering of concrete become more failure on time dependent of probability.

4.4 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. Figure 4.16 and 4.17 shows the priority of maintenance due 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 Figure 4.18, the results that G3 and G5 are the most concern because its failure increased very fast.

![Figure 4.16 Ranking of failure based on corrosion initiation](image.png)
The second, the tendency of failure due to serviceability limit states results on allowable stress of prestressing and concrete and indicated that the allowable stress of prestressing is more concerned than allowable stress of concrete as shown in Figure 4.18 and 4.19. G3 and G5 are still severe failure according to violets of corrosion and less of covering depth of concrete respectively.
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 Figure 4.21. 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 Figure 4.22.

It can be summarized that the priority of maintenance decision can be applied for maintenance planning of bridge. The technique is intended to help decision making process on the necessity of maintenance, repair and rehabilitation of bridges.
Figure 4.21 Ranking of failure based on flexural strength

Figure 4.22 Ranking of failure based on shear strength
Chapter 5
Conclusions and Recommendations

5.1 Conclusion

Development of a model for evaluation of structure and development of reliability of structure base on probability function in term of software for maintenance planning support are compromised to the objective of this study.

The deterioration prediction model developed in this study had primary objective of being a suitably simple model though preserving the accuracy of results. The proposed models are conformed to standard of prestressed concrete design and durability and service life of concrete structure in both of domestically and internationally standard for suitability.

Reliability of structure based on probability demonstrates that it can be improve the accuracy of deterioration prediction of the structures. The analysis aids the user of the software to identify how each parameter influences the performance of prestressed concrete bridge girder. So, It can be concluded that:

- The most influential variables were the chloride concentration, chloride diffusion, corrosion rate, carbonation concentration and covering of concrete.
- The durability criterion is the most significant consideration following by serviceability and ultimate criteria respectively.

Additional advantage, priority of maintenance decision can be applied in order to support for preventive maintenance, risk management and limited budget.

5.2 Recommendation for future studies

As recommendation for future studies, continuous developments can be created on both of deterioration prediction model and the software. Future work is need in improving the user-friendly interface of the software. Update model should improve both of structure analysis and deterioration prediction with time.
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Appendices
Appendix A
User Manual of Software

The explanation in this section follows step by step the procedure when performing an analysis. Figure A-1 illustrates the introduction and main window of the software. The software is comprised of three different options:

1) Data for input the variables
2) Perform a simulation for predicting the performance of structure on their criteria.
3) Export data to Microsoft excel for convenient analysis

Figure A-1 Introduction window

A1. Input data

In the input option, the general information is introduced along with the model parameters as shown in Figure A-2. This optional window is consist of three kinds of selection: create an input information for generating the data files to
simulation, edit an existing information for revising the previous data files and view former input information to observe any existing data files.

In general information is comprised of the project name, route, location of beam in term of station and global positioning system (GPS), construction years, date on generated data files and the important information of simulation like time to prediction and number of iteration for Monte Carlo simulation as shown in Figure A-3. The section of concrete window, the dimension of I girder is introduced by type of girder as well as girder length (mm) and spacing of girder (mm) as shown Figure A-4.
The prestressing strand window is where the user defines the information about specification of prestressing strand. Firstly, K value is dependent on the type of strand used, 0.28 for low relaxation strand, 0.38 for stress relieved strand type I and 0.48 for stress relieved strand type II. Next, characteristic of strand conform to specification of AASHTO in grade 250 and 270. Then, the locating of prestressing strand are described such as harping point (m), covering of prestressing strand (mm), the distance from centroid to bottom of section at mid span and end span. Finally, strand size and number of strand are used as shown Figure A-5.
Figure A-6 Reinforcing steels window

The reinforcing steel window is where the user identifies the data related to the characteristic of reinforcing steel such as diameter and spacing (mm) and covering of concrete (mm) as shown Figure A-6. All material properties shall key on material properties window as shown Figure A-7. Firstly, concrete properties are defined such as compressive strength of concrete (MPa) and weight of concrete (kg/m³). Next, prestressing strands properties are expressed on tensile strength (MPa) and modulus of elasticity (MPa). Finally, steel properties are identified on yield strength (MPa) and modulus of elasticity (MPa).

Figure A-7 Material properties window
The environmental condition window, relative humidity (%), temperature (Celsius), rain subjection, carbon dioxide concentration (ppm) and the location of structure from seashore (m) are presented as shown Figure A-8. Environment condition and inspection results are required such as carbonation coefficient (mm/√year), chloride diffusion (cm²/year), chloride concentration at surface of concrete (kg/m³), corrosion rate (µA/cm²), strength and covering of concrete (mm) for inspection results window on Figure A-9. Moreover mix proportion (kg) and time of deck placement (day) is represented on other window as shown Figure A-10.

Figure A-9 Results of inspection
A2 Simulations

There are two kinds of deterioration prediction which comprised with average years failure prediction and probability of failure according to Monte Carlo simulation as shown on Figure A-11. To perform a simulation, choose a project name that need to analysis. In average value is determined for time to failure due to their criteria in term of years as presented on Figure A-12.
Figure A-12 Deterioration predictions in average value window

In probability of failure, the software performs the model simulation according to Monte Carlo simulation which considerable depending on many factors such as number of iteration, number of time prediction, among other. The software displays the results in graphs on durability, serviceability and ultimate. Figure A-13 is an illustration of graphs presented.

Figure A-13 Probability of failure window
A3 Export data

Once the simulation has ended, the results are always displayed in the pattern of a single graph only in software. In order to take a better convenient to analyze the simulation output for comparison of results from various analyses, it can be converting the results from software into MS Excel as presented on Figure A-14.

Figure A-14 Export data to Microsoft Excel
Appendix B
Installation CD

B1. Contents of installation CD

Directory install with the set up files for the probabilistic deterioration prediction of prestressed concrete bridge girder software.